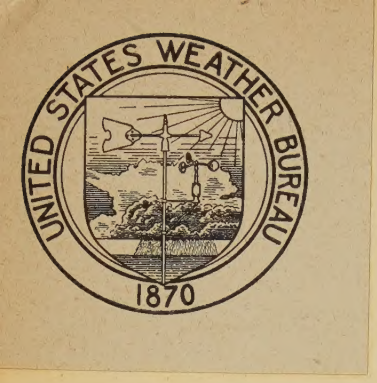

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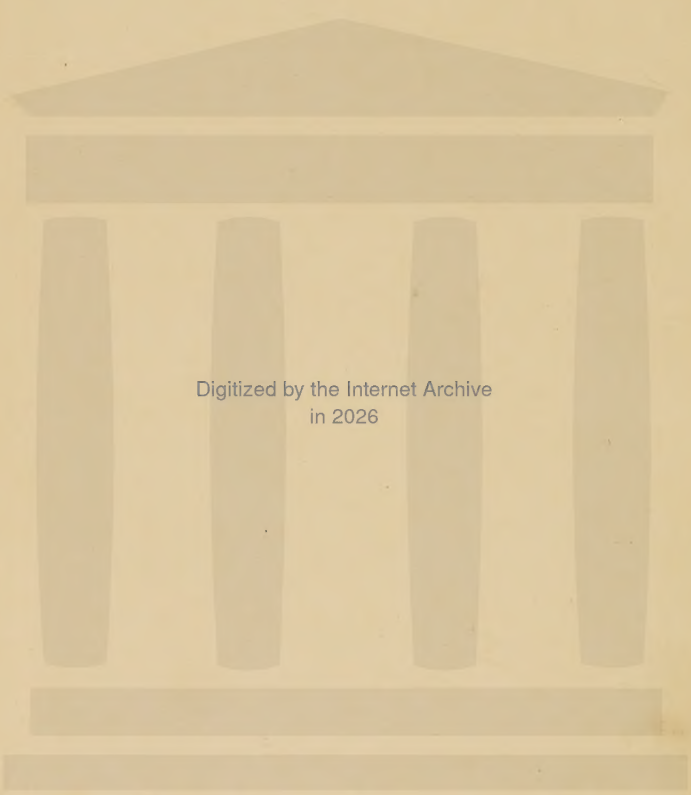

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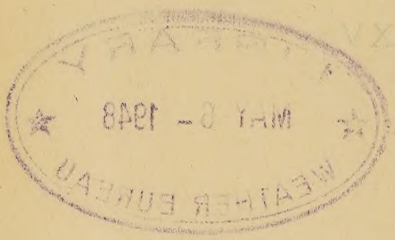

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FREDERIC BORRADAILE PRICHETT, JUN. AM. SOC. C. E. ... 1792

PLATES

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ERRATA

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Page 284, Fig. 18 (*a*) and Fig. 18 (*b*), for "0.5", vertical component at top of mast, read "0.25"; and for "0.5 + 0.782 = + 1.282", mast stress, read "0.25 + 0.782 = + 1.032".

Page 389, line 2, for "as an embankment requires no more than will keep the rails in line", read "as on embankments, and it will require no more anchoring than will keep the rails in line."

Page 529, Equation (4), for "$\frac{R}{l} = \rho\, v^2\, F\left(\frac{v\, d\, \rho}{\mu}\right)$", read "$R = \rho\, v^2\, F\left(\frac{v\, d\, \rho}{\mu}\right)$".

Page 530, Equation (7), for "$\frac{h}{l} = \frac{C v}{\gamma\, g\, d^2}$", read "$\frac{h}{l} = \frac{C v \mu}{\gamma\, g\, d^2}$".

Page 534, Equation (8), for "$\frac{h}{l} = \frac{0.1580\, v^{1.75} \left(\frac{\mu}{\rho}\right)^{0.25}}{\gamma\, g\, d^{1.25}}$",

read "$\frac{h}{l} = \frac{0.1580\, v^{1.75}\, \rho \left(\frac{\mu}{\rho}\right)^{0.25}}{\gamma\, g\, d^{1.25}}$".

Transactions, Vol. LXXXV

Page 120, footnote, line 3 from bottom, for "inadvertably," read "inadvertently."

Page 195, line 23, for "area" read "era".

Page 546, footnote, line 1, for "Ecology", read "Œcology".

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

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Paper No. 1480

THE FLOOD OF JUNE, 1921, IN THE ARKANSAS RIVER, AT PUEBLO, COLORADO*

By James Munn† and J. L. Savage,‡ Members, Am. Soc. C. E.

With Discussion by Messrs. Arthur O. Ridgway, R. G. Hosea, George G. Anderson, Robert Follansbee, and E. E. Jones.

Synopsis

This paper describes the causes and effects of the flood of June, 1921, in the Arkansas River, at Pueblo, Colo., and discusses general plans and estimates for future flood-control works.

A history of former floods is followed by a description of the recent flood, including a discussion of the causes, the resulting property damage, the estimated peak flow and flood volume, the drainage area and rainfall data, and a presentation of alternative plans and estimates for flood-control works.

History of Former Floods

The first flood in the Arkansas Valley known to white settlers occurred in 1864. At that time, Pueblo was little more than a trading post, and the damage was slight. The next flood of unusual volume occurred in 1894. At the time of this flood, Pueblo had little or no river protection, and the Arkansas River meandered through the city, cutting its banks and changing its course. This flood did considerable damage by covering the railroad yards and flooding the city to Third and Fourth Streets. After the flood of 1894, the river channel was straightened and substantial levees were built, leaving the river in the condition obtaining at the time of the flood of June, 1921.

With the exception of a flood in the Purgatoire River, a tributary of the Arkansas, in 1908, which washed out the Fort Bent Canal diversion dam and the Amity Canal diversion dam, there has been little damage to irrigation or

* Presented at the meeting of October 5th, 1921.

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other works through floods on the Arkansas River or its tributaries in the past twenty years. The minor damages which have occurred from time to time during this period, have been due more to poor construction or insufficient protection than to unusual flood conditions.

Description of the June, 1921, Flood

On the afternoon of June 2d, 1921, the Arkansas River at Pueblo was carrying 8 100 sec-ft. At 11.30 P. M., the river began to rise rapidly and, at 2.00 A. M., on June 3d, the discharge was about 28 500 sec-ft. At 8.00 A. M., June 3d, the discharge had dropped to 3 500 sec-ft. and, from noon to 5.00 P. M., it again began to rise rapidly, overtopping the levees and beginning to rise very rapidly, reaching a gauge height of 12.7 ft. and a discharge of 24 000 sec-ft. at 6.40 P. M., where it remained stationary until 7.40 P. M. At 7.40 P. M., it again began to rise rapidly, overtopping the levees and beginning to flood the city at 8.45 P. M., with a gauge height of 18.14 ft.

When the river began to overflow its banks, the discharge was probably about 40 000 sec-ft., but from the time of overflow the quantity of water passing through the city cannot be accurately determined, due to the choking of the channel with débris of all kinds. Subsequent levels showed a maximum gauge height of 24.66 ft., and the peak discharge has been roughly estimated at 100 000 sec-ft. The river after overflowing at 8.45 P. M., on June 3d, continued to rise until about 1.30 A. M., of June 4th, when it began to recede. At 4.30 A. M., it had fallen to a gauge height of about 18 ft., with an estimated discharge of about 50 000 sec-ft.

Sometime during the night of June 3d, a flood came down Fountain Creek, a tributary from the north, which joins the Arkansas River at Pueblo. The peak of this flood has been roughly estimated at 50 000 sec-ft. Although this flood receded quickly, it did considerable damage along its own course and added greatly to the damage in the Arkansas Valley below Pueblo.

On Sunday, June 5th, at about 3.00 P. M., another flood in the Arkansas River swept through Pueblo, adding somewhat to the damage and causing renewed alarm. This flood was caused by the destruction of the Schaeffer Dam on Beaver Creek, which released about 3 100 acre-ft. of reservoir storage. Probably no damage would have resulted from this flood if the levees had not already been breached by the greater flood of June 4th. In this connection, it will be noted that the flood of June 5th, resulting from the destruction of the Schaeffer Reservoir, totaled only 3 100 acre-ft., or about one-thirtieth of the whole flood volume.

The flood in the Arkansas River below its junction with Fountain Creek at Pueblo was augmented to a considerable extent by floods in some of the tributaries entering below Pueblo. The St. Charles River added probably 10 000 sec-ft., and this stream did considerable damage along its own course. At La Junta, Colo., the peak in the Arkansas River was probably between 170 000 and 175 000 sec-ft. Below La Junta, the accretions were negligible, and near Lamar, at the Amity Canal diversion dam, the peak flow was estimated at 170 000 sec-ft.

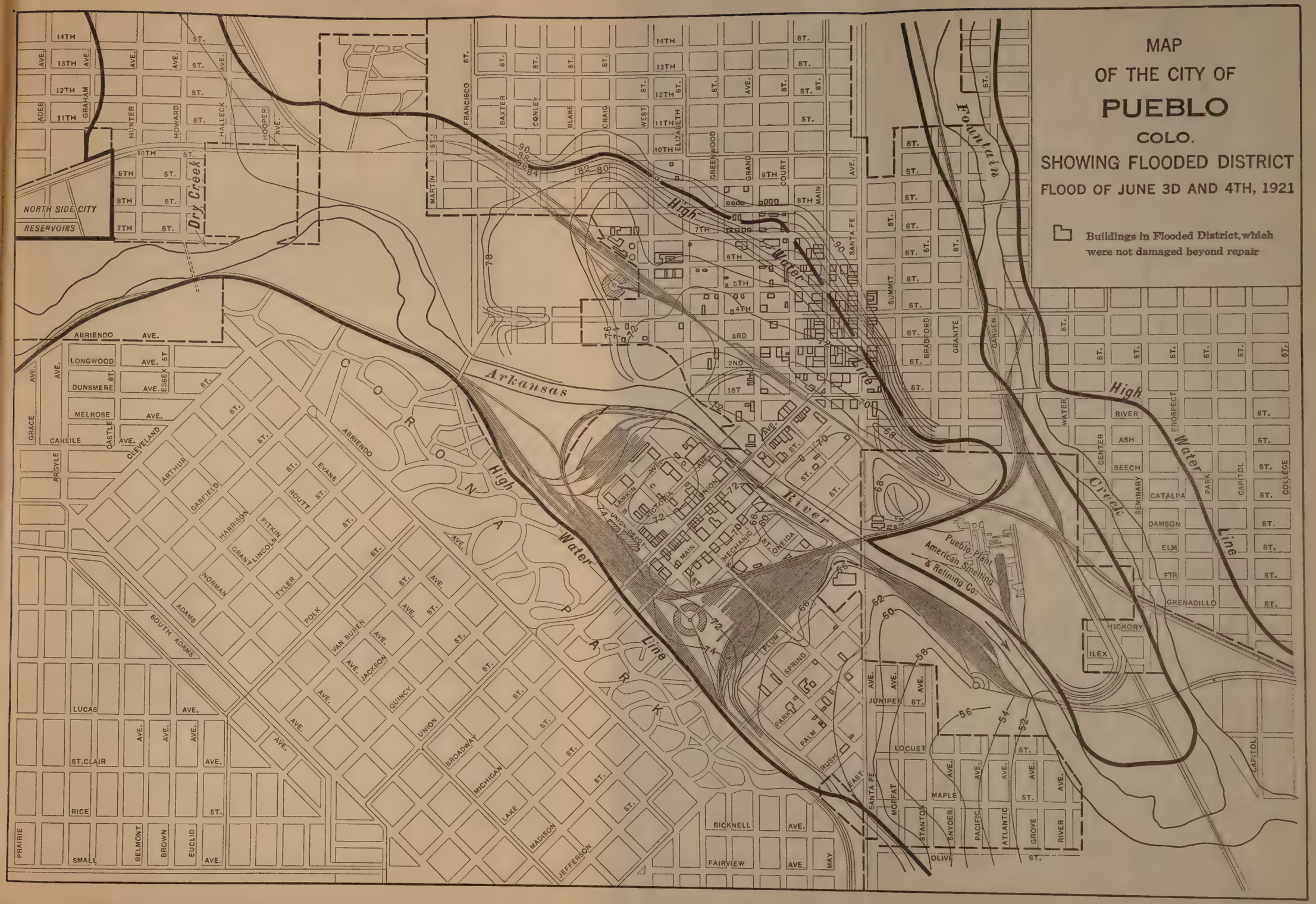
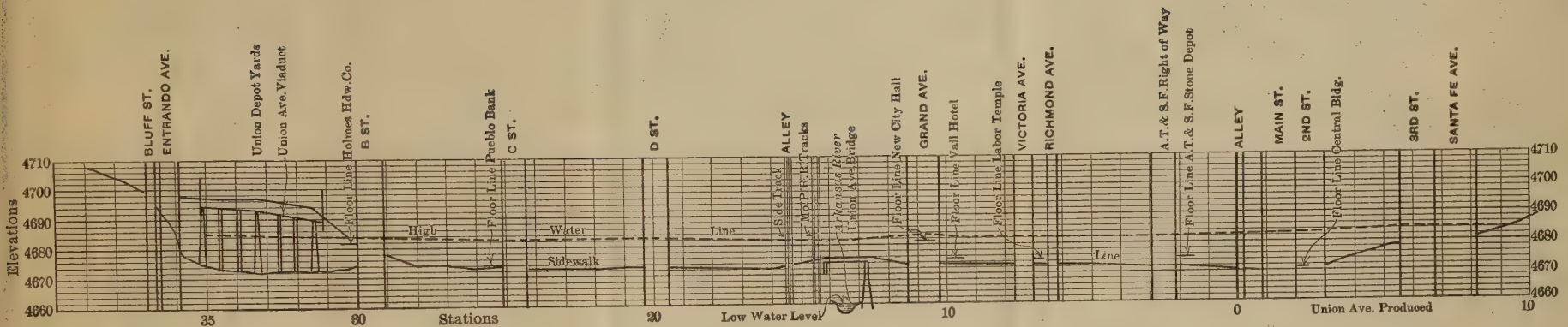
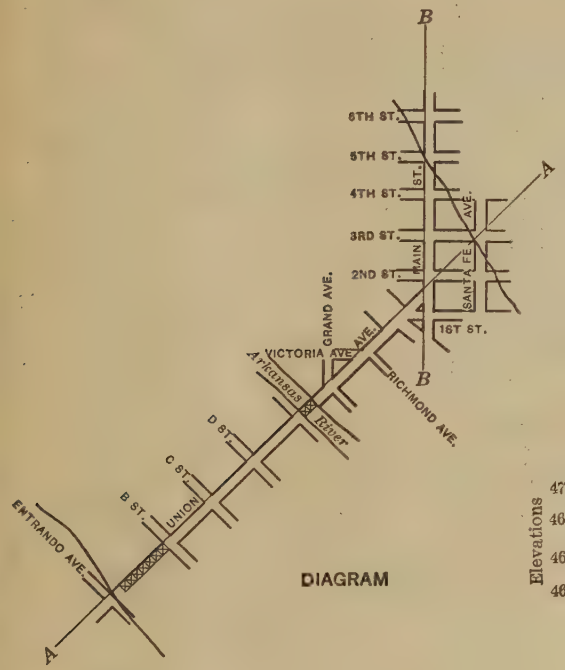

MAP
OF THE CITY OF
PUEBLO
COLO.
SHOWING FLOODED DISTRICT
FLOOD OF JUNE 3D AND 4TH, 1921
Buildings in Flooded District, which were not damaged beyond repair
NORTH SIDE CITY RESERVOIRS
Dry Creek
Arkansas
High Water Line
Fountain
Pueblo Plant American Smelting & Refining Co.
CORONA PARK
ABRIENDO AVE.
SOUTH ADAMS
14TH ST.
13TH AVE.
12TH ST.
11TH ST.
10TH ST.
9TH ST.
8TH ST.
7TH ST.
6TH ST.
5TH ST.
4TH ST.
3RD ST.
2ND ST.
1ST ST.
MAIN
UNION AVE.
SANTA FE
MIDLAND
UNION-DESPOT
VICTORIA
LAMKIN
MECHANIC
ONEIDA
PLUM
SPRING
PARK
PALM
RUSH
EAST
JUNIPER
LOCUST
MAPLE
OLIVE
MORFAT
STANTON
SNYDER
PACIFIC
ATLANTIC
GROVE
RIVER
WATER
CENTER
BEECH
ASH
CATALPA
DAMSON
ELM
FIR
GRENADILLO
HICKORY
ILEX
CAPITOL
PROSPECT
SEMINARY
PARK
COLLEGE
SUMMIT
BRADFORD
GRANITE
GARDEN
LONGWOOD
DUNSMERE
MELROSE
CARLILE
GRACE
ARGYLE
CASTLE
CLEVELAND
ESSEX
ARTHUR
GARFIELD
HARRISON
PITKIN
GRANT
LINCOLN
EVANS
ROUTT
NORMAN
TYLER
POLK
VAN BUREN
JACKSON
QUINCY
UNION
BROADWAY
MICHIGAN
LAKE
MADISON
JEFFERSON
LUCAS
ST. CLAIR
RICE
SMALL
PRAIRIE
BELMONT
BROWN
EUCLID
BICKNELL
FAIRVIEW
MAY
ADAMS
ADEE
GRAHAM
HUNTER
HOWARD
HALLECK
HOOPER
FRANCISCO
BAXTER
CONLEY
BLAKE
CRAIG
WEST
ELIZABETH
GREENWOOD
GRAND
COURT
MARTIN

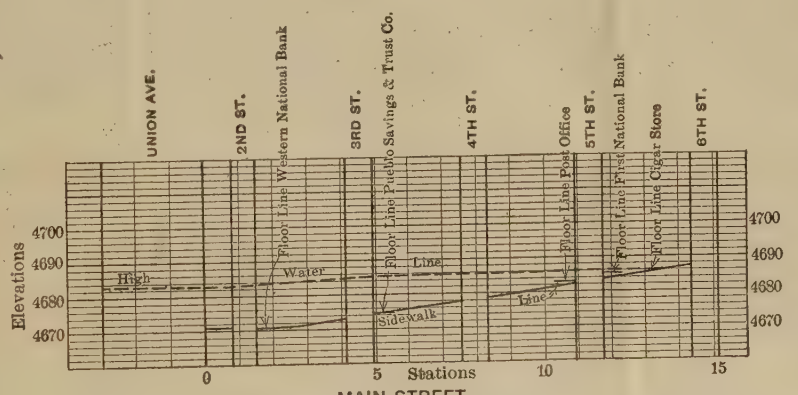
PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXV, No. 1480.
MUNN AND SAVAGE ON
FLOOD OF JUNE, 1921, AT PUEBLO, COLO.



UNION AVENUE
(BLUFF STREET TO SANTA FE AVE.)
LINE *A-A*



DIAGRAM



MAIN STREET
(UNION AVE. TO 6TH STREET)
LINE *B-B*

PROFILES
ACROSS FLOODED DISTRICT
CITY OF
PUEBLO
COLORADO
FLOOD OF JUNE 3RD AND 4TH, 1921

FIG. 1.—AEROPLANE VIEW OF FLOOD OF JUNE, 1921, IN THE ARKANSAS RIVER AT PUEBLO, COLO.

FIG 2.—VIEW OF MICKALLS' PACKING PLANT, AND ARKANSAS RIVER, PUEBLO, COLO.

FIG. 2.—AEROPLANE VIEW OF FLOOD DAMAGE IN RAILROAD YARDS, PUEBLO, COLO.

Although the flood peaks, as estimated, were very high, the duration of these high peaks was not long and, consequently, the flood volumes were not as large as might be expected. Rough estimates indicate that a total volume of about 100 000 acre-ft. passed through Pueblo from the Arkansas River and that from about 50 000 to 75 000 acre-ft. was added by the Fountain, St. Charles, and other streams down the valley, during the period of the flood.

Flood Losses

Preliminary estimates indicate that the property losses will total as follows:

Federal, State, and county	$900 000
Municipal	800 000
Real estate (city and town)	3 420 000
Personal (city and town)	3 575 000
Farm	3 675 000
Irrigation works	1 275 000
Railroads	4 685 000
Public utilities	500 000
Other property not included above	250 000
Total actual damage	$19 080 000

In addition to the damages, as estimated in the foregoing, there are intangible losses which are not covered in the estimates. The loss to railroads, public utilities, manufacturers, merchants, and business concerns generally, from the suspension of business, will aggregate a large sum. Likewise, the depreciation of property values, both within and without the flooded districts, if susceptible of estimate, would add materially to the actual losses.

In connection with the depreciation of property values, it should be noted that the ultimate depreciation will depend to a large extent on the scope of the flood-prevention measures finally adopted. If heroic measures are undertaken, and the City of Pueblo and other interests down the valley are made safe against future floods, the ultimate depreciation of property values will be comparatively little. However, if adequate flood protection is not provided, the depreciation of values will undoubtedly be very great.

The City of Pueblo suffered an actual loss of about $10 000 000 out of the total of $19 080 000. The effect of this loss can be realized by comparing it with the city's assessed valuation of $33 000 000. Besides suffering these staggering losses, the city was left with an expensive clean-up job. Fortunately, the War Department undertook extensive sanitary work, including the cleaning away of mud and débris and the repairing of damage to the city water-works system, furnishing the funds and equipment, in addition to a company of engineers, for such work.

Estimated Peak Flow and Volume

Probably the best estimate of the peak flow and the volume of the flood that can be made without extensive field surveys, is contained in a preliminary report to the State Engineer of Colorado, by R. G. Hosea, Deputy State Engineer. This report contains a table which is reproduced herein as Table 1.

TABLE 1.—QUANTITATIVE ESTIMATE OF PUEBLO FLOOD.

Date, 1921.	Time.	Gauge.	Second-feet.	Average second-feet.	Hours.	Acre-feet.
June 2	5.30 P. M.	7.8	7 900			
				8 100	6	4 050
June 2	11.30 P. M.	8.0	8 300			
				18 400	2½	3 833
June 3	2.00 A. M.	13.7	28 500			
				16 000	6	8 000
June 3	8.00 A. M.	5.5	3 500			
				3 150	4	1 050
June 3	12.00 M.	5.0	2 800			
				2 800	5	1 160
June 3	5.00 P. M.	5.0	2 800			
				10 350	1⅛	1 000
June 3	6.00 P. M.	11.2	17 900			
				20 950	½	873
June 3	6.40 P. M.	12.7	24 000			
				24 000	1	2 000
June 3	7.40 P. M.	12.7	24 000			
				34 500	1	2 875
June 3	8.30 P. M.	16.85 (Max. channel could carry)	45 000			
June 3	8.45 P. M.	overflow		72 500	5	30 200
June 4	1.30 A. M.	Max. (Est. from reports)	100 000 (Est.)	75 000	3	18 750
June 4	4.30 A. M.	18.0	50 000 (Est.)	45 000	4	22 500
Total						96 296

The main part of the flood started at 5.00 P. M. on June 3d, and totaled more than 78 000 acre-ft. in the following 18 hours. At 10.30 A. M., June 4th, the river was still flowing 40 000 sec-ft., as estimated by Mr. Hosea. There is, however, no information on which to base an accurate estimate of the volume added after this time. It is probable that fully 20 000 acre-ft. was added and that the main part of the flood, starting at 5.00 P. M., June 3d, and ending some time during the night of June 4th, totaled about 100 000 acre-ft. Rough estimates indicate that the peak flow in Fountain Creek was about 50 000 sec-ft., and that the flood volume was about 50 000 acre-ft. A hydrograph of the flood in the Arkansas River has been plotted, corresponding with the data given in Table 1, which is shown on Fig. 8.

DRAINAGE AREA AND RAINFALL DATA

The drainage area which contributed to the flood in the Arkansas River through Pueblo, embraces 1 740 sq. miles, between Canon City and Pueblo. The drainage area tributary to Fountain Creek, which creek joins the Arkansas River at Pueblo, includes 930 sq. miles. These drainage areas consist largely of barren land ranging in elevation from 4 600 to 9 000 ft. above sea level. The areas at different elevations are given in Table 2.

The drainage area on the Arkansas River above Canon City, embracing 3 060 sq. miles, contributed very little to the flood, and, in estimating the run-off, this area can be eliminated from consideration. Although the rainfall was quite general over the tributary area of 1 740 sq. miles, the excessive rainfall was confined to a comparatively small part of this area. The total volume of water which passed Pueblo has been estimated at 100 000 acre-ft. and

FIG. 4.—WRECKAGE AT RAILROAD CROSSING OVER THE ARKANSAS RIVER.

FIG. 5.—WRECKAGE IN YARDS OF DENVER AND RIO GRANDE RAILWAY COMPANY. ARKANSAS RIVER IN BACKGROUND.

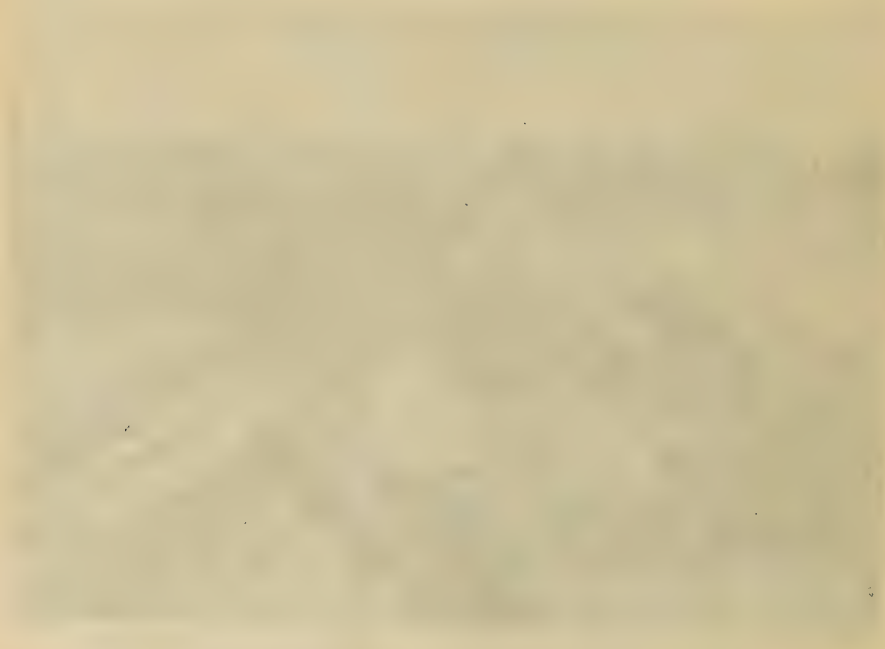


FIG. 6.—BRIDGE AND TRACK DESTRUCTION BY FOUNTAIN CREEK AT FIRST STREET, PUEBLO, COLO.

FIG. 7.—VIEW OF NORTH UNION AVENUE, PUEBLO, COLO., LOOKING TOWARD THE ARKANSAS RIVER.

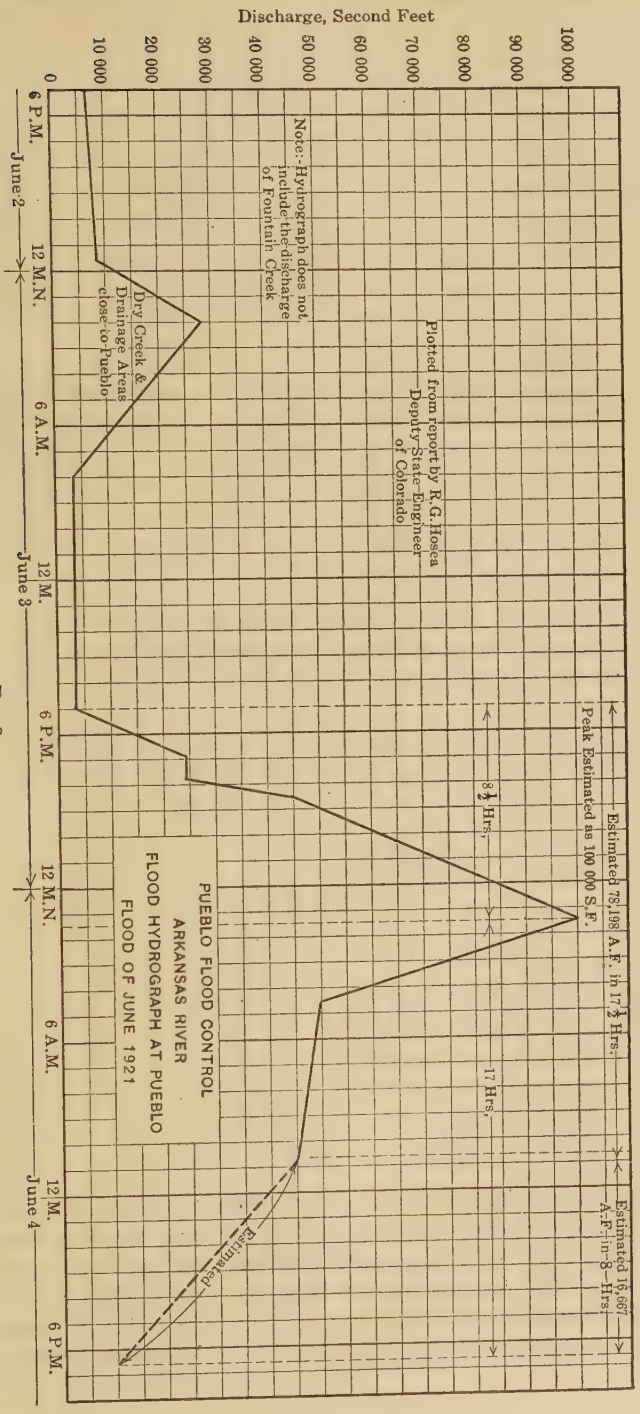

Discharge, Second Feet
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
6 P.M.
June 2
12 M.N.
6 A.M.
12 M.
June 3
6 P.M.
12 M.N.
6 A.M.
12 M.
June 4
6 P.M.
Estimated 78,198 A.F. in 17½ Hrs.
Peak Estimated as 100 000 S.F.
8½ Hrs.
17 Hrs.
Estimated 16,667 A.F. in 8 Hrs.
Plotted from report by R.G. Hosea Deputy State Engineer of Colorado
Note:-Hydrograph does not include the discharge of Fountain Creek
Dry Creek & Drainage Areas close to Pueblo
Estimated
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
FLOOD HYDROGRAPH AT PUEBLO
FLOOD OF JUNE 1921

FIG. 8.

TABLE 2.—Drainage Areas at Different Elevations.

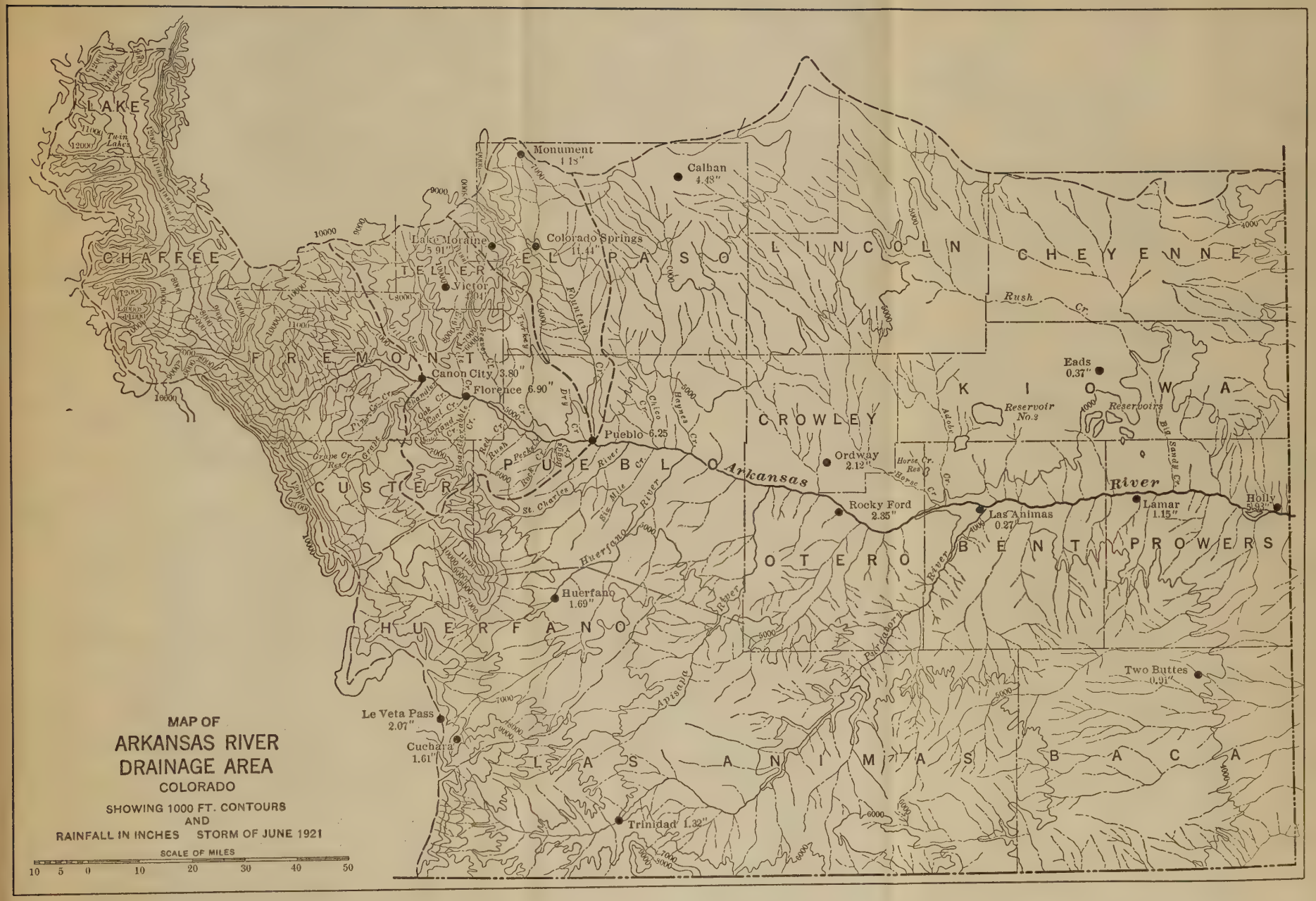
Arkansas River.		Fountain Creek.	
Elevation, in feet.	Area, in square miles.	Elevation, in feet.	Area, in square miles.
4 000 to 5 000	88	4 000 to 5 000	38
5 000 to 6 000	632	5 000 to 6 000	337
6 000 to 7 000	229	6 000 to 7 000	273
7 000 and over	791	7 000 and over	282
Total	1 740	Total	930

probably one-half this volume came from less than 300 sq. miles of drainage area between Hardscrabble Creek and Pueblo. In this respect, the storm which caused the flood was far from a maximum, and it is probable that a rainfall of an intensity equal to that which fell between Hardscrabble Creek and Pueblo might easily extend over an area of 1 000 sq. miles, resulting in a run-off of more than three times that of the recent flood. The drainage areas are shown in tabular form in Table 3 and in map form on Plate III and Fig. 9.

TABLE 3.—Drainage Area of Tributaries of Arkansas River, Between Canon City and Pueblo, and of Fountain Creek.

Arkansas River, from north:		
Oil Creek	456 sq. miles	
Six-Mile, Eight-Mile, and Birch Hollow Creeks	140 " "	
Beaver Creek	260 " "	
Turkey Creek	215 " "	
Dry Creek	71 " "	
Total from north		1 142 sq. miles
Arkansas River, from south:		
Chandler Creek	38 sq. miles	
Oak Creek	72 " "	
Coal Creek	28 " "	
Hardscrabble Creek	186 " "	
Ritchie Gulch	40 " "	
Red Creek	42 " "	
Rush Creek	34 " "	
Peck Creek	46 " "	
Rock Creek	67 " "	
Boggs Creek	25 " "	
Small creeks between Boggs Creek and Pueblo	20 " "	
Total from south		598 " "
Total between Canon City and Pueblo		1 740 sq. miles
Fountain Creek		930 sq. miles

In studying the rainfall records of the recent storm, and of other storms in the Arkansas Valley, it is noted that in the two largest storms, namely, those of May, 1894, and June, 1921, the average rainfall increases quite uniformly with the elevation of the drainage area. This relation between rainfall and elevation of drainage area is shown in Table 6, from which the average rainfalls given in Table 7, are taken for the storms of May, 1894, and June, 1921.



MAP OF
ARKANSAS RIVER
DRAINAGE AREA
COLORADO

SHOWING 1000 FT. CONTOURS
AND
RAINFALL IN INCHES STORM OF JUNE 1921

SCALE OF MILES
10 5 0 10 20 30 40 50

TABLE 4.—Daily and Cumulated Rainfall over Arkansas Drainage Area in Colorado during the Storm of May, 1894.

(Order of Stations is from West Proceeding East)

Station.	Daily Rainfall, in Inches.						Cumulated Rainfall, in Inches.					Rainfall to:
	Day of month.						Days of month.					
	28	29	30	31	1	2	28-29	28-30	28-31	28-1	28-2	
Lake Moraine	.....	.....	5.50	2.00	.....	.....	.....	5.50	7.50	.....	.....	6.00 P. M.
Canon City	.....	0.75	4.31	.....	.....	.....	.....	5.06	.....	.....	.....	.....
Husted	0.03	0.08	1.15	0.84	0 22	.....	0.11	1.26	2.10	2.32	.....	6.00 A. M.
Glen Eyrie	.....	.....	3.13	1.58	0.15	.....	.....	3.13	4.71	4.86	.....	.....
Colorado Springs	.....	0.08	2.95	1.44	0.50	.....	0.08	3.03	4.47	4.97	.....	.....
Divide Exp. Station	.....	0.02	1.65	1.82	.....	.....	0.02	1.67	3.49	.....	.....	12.00 P. M.
Hamps	.....	0.06	.....	2.70	.....	.....	0.06	0.06	2.76	.....	.....	7.00 P. M.
Rocky Ford	.....	.....	.....	3.50	.....	.....	.....	.....	3.50	.....	.....	.....
Las Animas	.....	.....	0.07	1.09	.....	.....	.....	0.07	1.16	.....	.....	7.00 P. M.
Cheyenne Wells	.....	.....	2.00	.....	.....	.....	.....	2.00	.....	.....	.....	.....
Springfield	.....	4.00	.....	0.10	.....	.....	4.00	4.00	4.10	.....	.....	.....
Vilas	0.14	0.62	0.93	0.76	.....	.....	0.76	1.69	2.45	.....	.....	.....

TABLE 5.—Daily and Cumulated Rainfall over Arkansas Drainage Area in Colorado during the Storm of June, 1921.

(Order of Stations is from West Proceeding East)

Station.	Daily Rainfall, in Inches.						Cumulated Rainfall, in Inches.					Rainfall to:
	Day of month.						Days of month.					
	2	3	4	5	6	7	2-3	2-4	2-5	2-6	2-7	
Victor	.....	0.03	2.08	1.55	0.37	0.01	0.03	2.11	3.66	4.03	4.04	4.00 P. M.
Canon City	.....	0.30	2.35	0.75	0.40	.....	0.30	2.65	3.40	3.80	.....	.....
La Veta Pass	.....	0.98	0.89	.....	0.20	.....	0.98	1.87	1.87	2.07	.....	.....
Lake Moraine	.....	0.65	3.68	1.40	0.18	.....	0.65	4.33	5.73	5.91	5.91	3.30 P. M.
Florence	.....	0.99	3.31	2.47	0.13	.....	0.99	4.30	6.77	6.90	.....	.....
Monument	0.06	.....	2.90	0.82	.....	0.40	0.06	2.96	3.78	3.78	4.18	4.00 P. M.
Colorado Springs	0.35	5.00	4.40	1.26	0.42	0.01	5.35	9.75	11.01	11.43	11.44	12.00 M.
Pueblo	1.94	1.64	1.45	1.12	0.09	0.01	3.58	5.03	6.15	6.24	6.25	3.25 P. M.
Huerfano	.....	.....	1.06	0.56	0.04	0.03	.....	1.06	1.62	1.66	1.69	6.30 P. M.
Cuchara Camps	.....	.....	0.86	0.21	0.12	0.42	.....	0.86	1.07	1.19	1.61	12.00 M.
Trinidad	.....	0.20	0.55	0.30	.....	0.27	0.20	0.75	1.05	1.05	1.32	3.15 P. M.
Calhan	.....	.....	3.26	0.83	0.39	.....	.....	3.26	4.09	4.48	.....	7.30 P. M.
Ordway	0.25	.....	0.90	0.75	0.19	0.03	0.25	1.15	1.90	2.09	2.12	.....
Rocky Ford	.....	.....	1.40	0.80	0.15	.....	.....	1.40	2.20	2.35	.....	.....
Las Animas	.....	0.27	.....	.....	.....	.....	0.27	.....	.....	.....	.....	8.00 P. M.
Eads	.....	0.03	0.13	0.21	.....	.....	0.03	0.16	0.37	.....	.....	5.00 P. M.
Lamar	.....	.....	0.50	.....	0.65	.....	.....	0.50	.....	1.15	.....	5.00 P. M.
Two Buttes	.....	.....	0.22	0.15	0.30	0.24	.....	0.22	0.37	0.67	0.91	.....
Holly	5.88	.....	0.05	.....	.....	.....	5.88	5.93	.....	.....	.....	.....

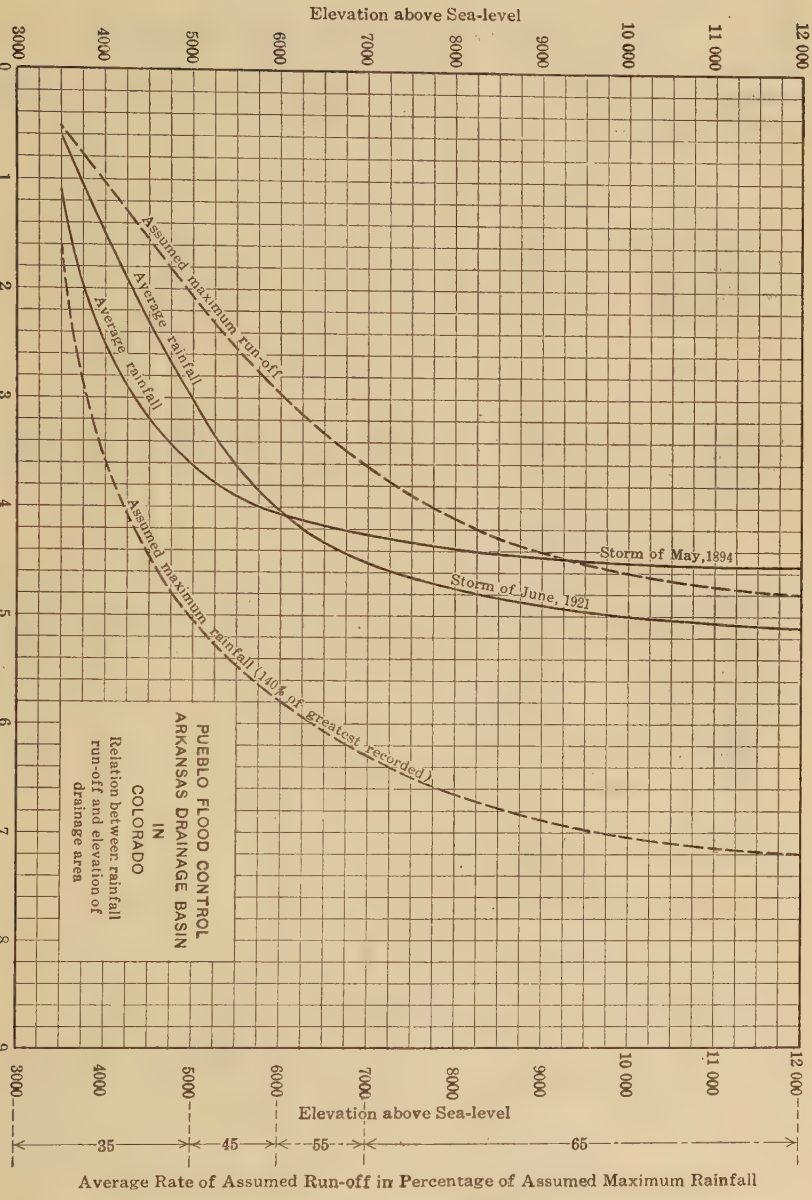

Elevation above Sea-level
3000
4000
5000
6000
7000
8000
9000
10 000
11 000
12 000
0
1
2
3
4
5
6
7
8
9
Rainfall in inches in 72 Hours.
Assumed maximum run-off
Average rainfall
Average rainfall
Storm of May, 1894
Storm of June, 1921
Assumed maximum rainfall (140% of greatest recorded)
PUEBLO FLOOD CONTROL
ARKANSAS DRAINAGE BASIN
IN
COLORADO
Relation between rainfall run-off and elevation of drainage area
Elevation above Sea-level
35
45
55
65
Average Rate of Assumed Run-off in Percentage of Assumed Maximum Rainfall

Fig. 10.

TABLE 6.—DEPTH OF RAINFALL, IN INCHES, FOR VARIOUS

Elevation, in feet.	Station.	May, 1894.			July, 1899.			June, 1903.		
		24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.
3 000 to 4 000	Holly	.....	.....	.....	1.16	1.16	1.16	3.25	3.92	4.19
	Lamar	.....	.....	.....	2.80	2.40	2.49	1.00	1.86	2.20
	Las Animas	1.09	1.16	1.16	1.95	2.07	2.07	1.20	1.60	1.60
	Average	1.09	1.16	1.16	1.80	1.88	1.91	1.82	2.46	2.66
4 000 to 5 000	Cheyenne Wells	2.00	2.00	2.00	0.37	0.52	0.69	0.25	0.40	0.53
	Ordway	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Pueblo	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Rocky Ford	3.50	3.50	3.50	2.08	2.55	2.70	1.65	1.74	1.90
	Springfield	4.00	4.00	4.10	.....	.....	.....	.....	.....	.....
	Two Buttes	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Average	3.17	3.17	3.20	1.23	1.54	1.70	0.95	1.07	1.22
5 000 to 6 000	Canon City	4.31	5.06	5.06	0.45	0.75	0.79	0.64	0.89	1.37
	Florence	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Hamps	2.00	2.70	2.76	0.18	0.32	0.36	0.92	1.84	2.18
	Hoenne	.....	.....	.....	0.34	0.51	0.84	1.31	1.81	1.96
	Trinidad	.....	.....	.....	0.66	0.87	1.01	1.10	1.59	2.35
	Average	3.16	3.88	3.91	0.41	0.61	0.75	0.99	1.53	1.97
6 000 to 12 000	Calhan	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Colorado Springs	2.95	4.39	4.47	1.03	1.18	1.29	1.27	2.13	2.41
	Cuchara Camps	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Divide Exp. Station	1.65	3.47	3.49	.....	.....	.....	.....	.....	.....
	Glen Eyrie	3.13	4.71	4.86	0.49	0.85	0.85	0.72	1.17	1.45
	Husted	1.15	1.99	2.07	.....	.....	.....	0.75	1.50	1.65
	Lake Moraine	5.50	7.50	7.50	1.36	1.60	1.72	1.00	1.65	2.05
	Monument	.....	.....	.....	.....	.....	.....	.....	.....	.....
	North Lake	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Salida	.....	.....	.....	0.90	1.32	1.49	0.48	0.76	1.10
	Santa Clara	.....	.....	.....	1.08	2.13	2.73	1.63	3.00	3.97
	Victor	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Wortman	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Average	2.88	4.41	4.48	0.97	1.42	1.62	0.98	1.70	2.10
3 000 to 12 000	General average	2.84	3.68	3.72	1.02	1.30	1.44	1.14	1.72	2.06

TABLE 7.

Elevation, in feet.	Average Rainfall, in Inches, in 72 Hours.	
	May, 1894.	June, 1921.
3 000 to 4 000	1.16	0.71
4 000 to 5 000	3.20	2.27
5 000 to 6 000	3.91	3.74
6 000 to 12 000	4.48	4.96
Average, 3 000 to 12 000	3.72	3.43

STATIONS IN THE ARKANSAS DRAINAGE AREA IN COLORADO.

Elevation, in feet.	Station.	July, 1914.			August, 1916.			June, 1921.		
		24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.
3 000 to 4 000	Holly	0.70	0.86	1.06	1.94	2.14	2.14	.....	.....	.....
	Lamar	1.00	1.25	1.25	2.05	2.65	3.05	0.65	0.65	1.15
	Las Animas	0.77	1.17	1.17	2.15	2.25	2.27	0.27	0.27	0.27
	Average	0.82	1.09	1.16	2.05	2.35	2.49	0.46	0.46	0.71
4 000 to 5 000	Cheyenne Wells	0.20	0.20	0.20	1.48	1.48	1.48	.....	.....	.....
	Ordway	.....	.....	.....	.....	.....	.....	0.90	1.65	1.84
	Pueblo	1.25	1.84	1.94	1 39	2.08	2.08	1.64	3.09	4.21
	Rocky Ford	2.00	2.32	2.61	2.50	2.58	3.08	1.40	2.20	2.35
	Springfield	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Two Buttes	0.66	1.07	1.48	2.87	3.14	4.75	0.22	0.37	0.67
	Average	1.03	1.36	1.56	2.06	2.32	2.85	1.04	1.83	2.27
5 000 to 6 000	Canon City	1.10	1.60	1.75	0.90	1.08	1.08	2.35	3.10	3.40
	Florence	.....	.....	.....	.....	.....	.....	3.31	5.78	6.77
	Hamps	0.78	1.08	1.08	1.36	1.36	1.36	.....	.....	.....
	Hoenne	1.20	1.81	2.11	0.80	0.95	1.16	.....	.....	.....
	Trinidad	0.75	1.53	2.06	0.97	0.97	0.97	0.55	0.85	1.05
	Average	0.96	1.51	1.75	1.01	1.09	1.14	2.07	3.24	3.74
6 000 to 12 000	Calhan	.....	.....	.....	.....	.....	.....	3.26	4.09	4.48
	Colorado Springs	2 00	2.03	2.03	0.28	0.28	0.28	5.00	9.40	10.66
	Cuchara Camps	.....	.....	.....	.....	.....	.....	0.86	1.07	1.19
	Divide Exp. Station	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Glen Eyrie	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Husted	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Lake Moraine	0.44	0.77	1.17	0.85	1.43	1.75	3.68	4.33	5.73
	Monument	.....	.....	.....	.....	.....	.....	2.90	3.72	3.72
	North Lake	0.78	1.46	2.01	0.75	1.37	1.57	.....	.....	.....
	Salida	0.80	0.94	1.04	0.72	1.42	1.42	.....	.....	.....
	Santa Clara	.....	.....	.....	.....	.....	.....	.....	.....	.....
	Victor	.....	.....	.....	.....	.....	.....	2.68	3.63	4.00
	Wortman	0.40	0.60	0.70	1.00	1.00	1.00	.....	.....	.....
	Average	0.88	1.16	1.39	0.72	1.10	1.20	2.70	3.63	4.96
3 000 to 12 000	General average	0.93	1.28	1.48	1.37	1.64	1.84	1.94	2 95	3 43

Curves showing the average rainfall in 72 hours at different elevations of the Arkansas River drainage area will be found on Fig. 10 for the storms of May, 1894, and June, 1921. Fig. 10 also shows a curve of assumed maximum rainfall in 72 hours, based on 140% of the greatest recorded rainfall in the storms of May, 1894, and June, 1921. It also shows a curve of assumed maximum run-off based on the following assumed percentages of run-off for the different elevations of the drainage area:

Elevation.	Percentage of Run-Off.
3 000 to 5 000	35
5 000 to 6 000	45
6 000 to 7 000	55
7 000 and over	65

TABLE 8.—Probable Maximum Run-Off from Drainage Area of the Arkansas River Between Canon City and Pueblo (1 740 Sq. Miles), Based on the Storms of May, 1894, and June, 1921.

(See Fig. 10.)

Elevation above sea level, in feet.	Area.		Average Rainfall in 72 Hours.				Run-off.		
			Maximum recorded.		Probable maximum.				
	Square miles	Acres.	Inches.	Acre-feet.	Inches.	Acre-feet.	Assumed percentage of rainfall.	Based on maximum recorded rainfall, in acre-feet.	Probable maximum, in acre-feet.
4 000 to 5 000	88	56 320	3.20	15 018	4.50	21 120	35	5 256	7 392
Sub-total..	88	56 320	.....	15 018	.....	21 120	.....	5 256	7 392
5 000 to 6 000	632	404 480	3.91	131 793	5.50	185 387	45	59 307	83 424
Sub-total...	720	460 800	.....	146 811	.....	206 507	.....	64 563	90 816
6 000 to 7 000	229	146 560	4.32	52 762	6.05	73 891	65	29 019	40 640
Sub-total...	949	607 360	.....	199 573	.....	280 398	.....	93 582	131 456
7 000 to 12 000	791	506 240	4.96	209 246	6.96	293 619	65	136 010	190 852
Total...	1 740	1 113 600	.....	408 819	.....	574 017	.....	229 592	322 368*

Note.—The probable maximum rainfall is estimated to be 40% in excess of the greatest recorded rainfall during the storms of May, 1894, and June, 1921.

* Equivalent to 3½-in. run-off from 1 740 sq. miles.

Using the same storm area that contributed to the recent flood in the Arkansas River, namely, 1 740 sq. miles between Canon City and Pueblo, and applying the assumed maximum run-off rates, as shown on Fig. 10, to respective areas for different elevations of the storm area, a value of 322 000 acre-ft. has been obtained for the probable maximum run-off through Pueblo. The calculation of this quantity is shown in detail in Table 8. A run-off of 322 000 acre-ft. from 1 740 sq. miles is equivalent to an average run-off of 3½ in. Based on this total run-off, a hydrograph (Fig. 11) has been constructed of a shape similar to the hydrograph of the flood of June, 1921, which indicates a probable peak flow of about 168 000 sec-ft. in the Arkansas River through Pueblo. A similar study of the drainage area of Fountain Creek (Table 9) results in a probable maximum run-off of 164 000 acre-ft. and a probable peak flow of 110 000 sec-ft., as indicated on Fig. 12.

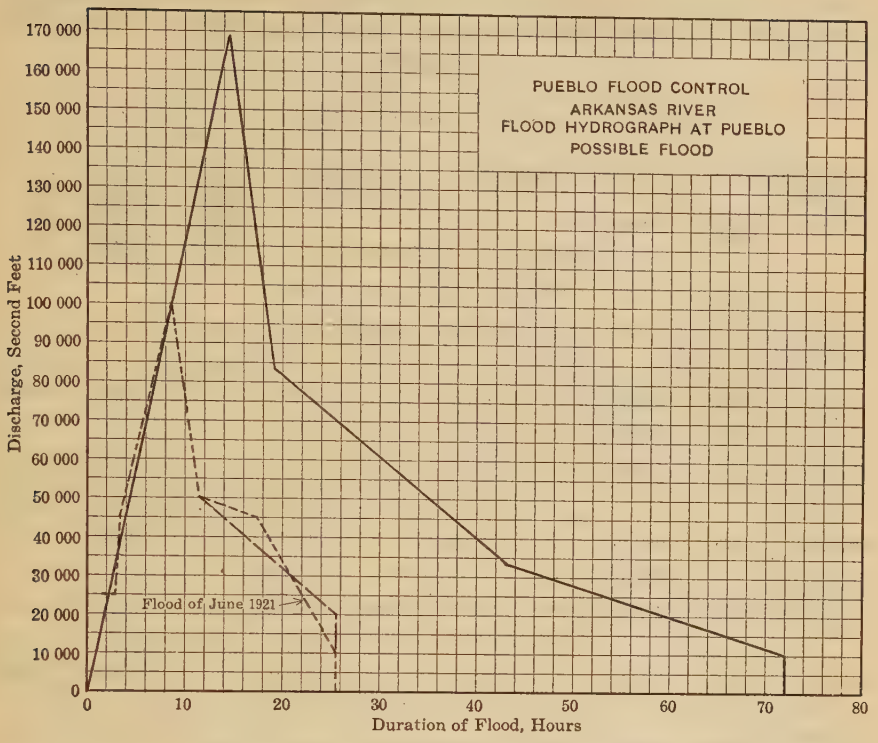

PUEBLO FLOOD CONTROL
ARKANSAS RIVER
FLOOD HYDROGRAPH AT PUEBLO
POSSIBLE FLOOD
Flood of June 1921
Discharge, Second Feet
Duration of Flood, Hours
170 000
160 000
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
0
10
20
30
40
50
60
70
80

FIG. 11.

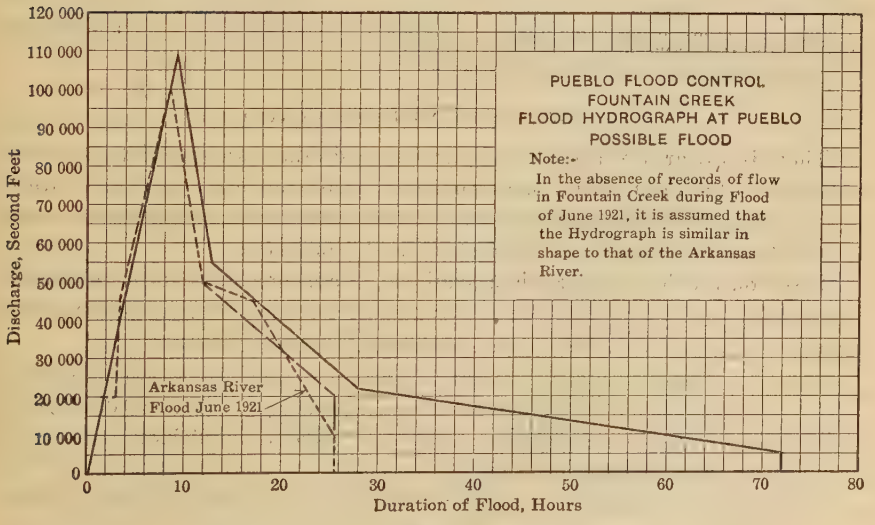

PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
FLOOD HYDROGRAPH AT PUEBLO
POSSIBLE FLOOD
Note:-
In the absence of records of flow in Fountain Creek during Flood of June 1921, it is assumed that the Hydrograph is similar in shape to that of the Arkansas River.
Arkansas River
Flood June 1921
Discharge, Second Feet
Duration of Flood, Hours
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
0
10
20
30
40
50
60
70
80

FIG. 12.

TABLE 9.—Probable Maximum Run-Off from Drainage Area of Fountain Creek (930 Sq. Miles), Based on the Storms of May, 1894, and June, 1921.

(See Fig. 10.)

Elevation above sea level, in feet.	Area.		Average Rainfall in 72 Hours,				Run-Off.		
			Maximum recorded.		Probable maximum.				
	Square miles.	Acres.	Inches.	Acre-feet.	Inches.	Acre-feet.	Assumed percentage of rainfall.	Based on maximum recorded rainfall, in acre-feet.	Probable maximum, in acre-feet.
4 000 to 5 000	38	24 320	3.20	6 485	4.50	9 120	35	2 270	3 192
Sub-total...	38	24 320	.....	6 485	.....	9 120	.....	2 270	3 192
5 000 to 6 000	337	215 680	3.91	70 276	5.50	98 853	45	31 624	44 484
Sub-total...	375	240 000	.....	76 671	.....	107 973	.....	33 894	47 676
6 000 to 7 000	273	174 720	4.32	62 899	6.05	88 088	55	34 594	48 448
Sub-total...	648	414 720	.....	139 660	.....	196 061	.....	68 488	96 124
7 000 to 12 000	282	180 480	4.96	74 598	6.96	104 678	65	48 478	68 041
Total..	930	595 200	.....	214 258	.....	300 739	.....	116 966	164 165*

Note.—The probable maximum rainfall is estimated to be 40% in excess of the greatest recorded rainfall during the storms of May, 1894, and June, 1921.

* Equivalent to 3½-in. run-off from 930 sq. miles.

Reconstruction and Flood Control

There is a dearth of accurate topography and other essential data on which to base even the most preliminary studies of flood-control works, and a careful investigation of alternative plans will involve considerable detail survey work before accurate designs and estimates can be prepared.

Based on the fragmentary data available, it appears that careful consideration should be given to three alternative plans for controlling or limiting the damage from future floods on the Arkansas River, as follows:

Plan A.—Flood-detention storage in sufficient capacity to limit the flow through Pueblo to the present channel capacity of about 25 000 sec-ft.;

Plan B.—Channel enlargement through Pueblo of sufficient capacity to pass the maximum peak flow;

Plan C.—A combination of flood-detention storage and channel enlargement.

Preliminary studies suggest the careful consideration of four alternative plans for limiting the damage from future floods in Fountain Creek, as follows:

Plan D.—Flood-detention storage;
Plan E.—Bank protection along both banks of the flood-plain;
Plan F.—Channel enlargement;
Plan G.—A combination of flood-detention storage with either bank protection or channel enlargement.

Flood Control on Arkansas River

Plan A.—A solution of the problem by flood-detention storage alone would involve the construction of one or more detention reservoirs of sufficient capacity to retard all waters in excess of the present channel capacity through Pueblo, estimated at 25 000 sec-ft. Based on the assumed maximum flood in the Arkansas River, represented in the hydrograph shown on Fig. 11, detention storage of 210 000 acre-ft. would be required to control the flow through Pueblo to 25 000 sec-ft.

The Rock Canyon Reservoir site, on the Arkansas River, about 8 miles west of Pueblo, is believed to be the best site on the river for a detention reservoir. This site is in a gorge in hard sandstone, which formation is known to be of great depth. The floor of the reservoir is privately owned and is partly under cultivation and partly in pasture. The tracks of the Denver and Rio Grande and Santa Fé Railroads, also the upper end of the Bessemer Canal, are in the reservoir site.

Utilization of this site would involve the reconstruction of these railroads for a distance of probably 8 to 10 miles and also some changes in the Bessemer Canal, unless it was considered safe to leave them in the reservoir area. In this connection, it is probable that a flood-detention storage reservoir with large permanent outlets could be utilized for the partial control of unusual floods without increasing the damage from such floods to the railroads or the Bessemer Canal in their present locations. In other words, the railroads and the Bessemer Canal are located so that they are subject to damage from great floods, the damage being principally due to high velocities in the river. With a flood-detention reservoir of large outlet capacity, all ordinary floods would pass the dam without storing water, and only the most unusual floods would submerge the railroads and the Bessemer Canal by storing water in the reservoir. The effect of submergence in comparatively quiet water might cause less damage than the high velocities, under present conditions.

It has been stated previously that a detention storage capacity of about 210 000 acre-ft. would be required to control the assumed maximum flood in the Arkansas River to a flow of 25 000 sec-ft. through Pueblo. Unfortunately, the available data indicate that the Rock Canyon Reservoir site cannot be economically developed to this capacity, and there are probably no other sites of suitable location and sufficient capacity to control the floods. It is concluded, therefore, that Plan *A* may not prove to be feasible, although it should be carefully considered when full data become available.

Fig. 13 shows a capacity curve and Fig. 14 a cost curve for the Rock Canyon Reservoir site from which the costs given in Table 10 are taken. Similarly, Figs. 15 and 16 show a capacity curve and a cost curve, respectively, for the Steel Hollow Reservoir site.

TABLE 10.—COSTS OF STORAGE IN ROCK CANYON RESERVOIR.

Height of dam, in feet.	Reservoir capacity, in acre-feet.	Estimated cost.
75	32 000	$2 000 000
100	61 000	4 000 000
125	100 000	6 500 000

It will be noted that storage in the Rock Canyon Reservoir site is estimated to cost about $65 per acre-ft. Storage on the tributaries, if available at all, would probably cost over 50% more than storage in the Rock Canyon Reservoir, or, say, $100 per acre-ft. On this basis, it might be estimated that 210 000 acre-ft. of flood-detention storage, if available, would cost approximately, as follows:

100 000 acre-ft., at $65	$6 500 000
110 000 acre-ft., at $100 (say)	11 000 000
Total	$17 500 000

Plan B.—A solution of the flood problem on the Arkansas River, based on channel enlargement through Pueblo alone (with no flood-detention storage), would involve the widening and deepening of the present river channel or the construction of other channels, in addition to extensive levee work. In the section of the paper dealing with the drainage area and rainfall data (page 10), it was shown that the peak flow resulting from the maximum assumed storm would be 168 000 sec-ft. This represents the required channel capacity through Pueblo under Plan *B*.

Preliminary studies based on incomplete data indicate that a concrete-lined channel (lined on both sides and the bottom), following in general the alignment of the present river channel, is likely to prove the most economical method of passing the peak flow through Pueblo. Such a channel with a capacity of 168 000 sec-ft., extending from above the Fourth Street Bridge to a point below the Missouri Pacific Bridge, together with the necessary levee construction, is estimated to cost $5 500 000, including excavation, concrete, new bridges, raising of railroad grades, and right of way, as the main items of cost.

Studies have been made on a smaller concrete-lined channel supplemented with grass channels, in the nature of sunken gardens, one on each side of the concrete channel. The additional cost for bridges and for right of way under this arrangement would probably bring the cost to about $7 000 000.

Under Plan *B*, the grade of all bridges should be raised to provide a minimum clearance of 10 ft. between the high-water surface and the lowest

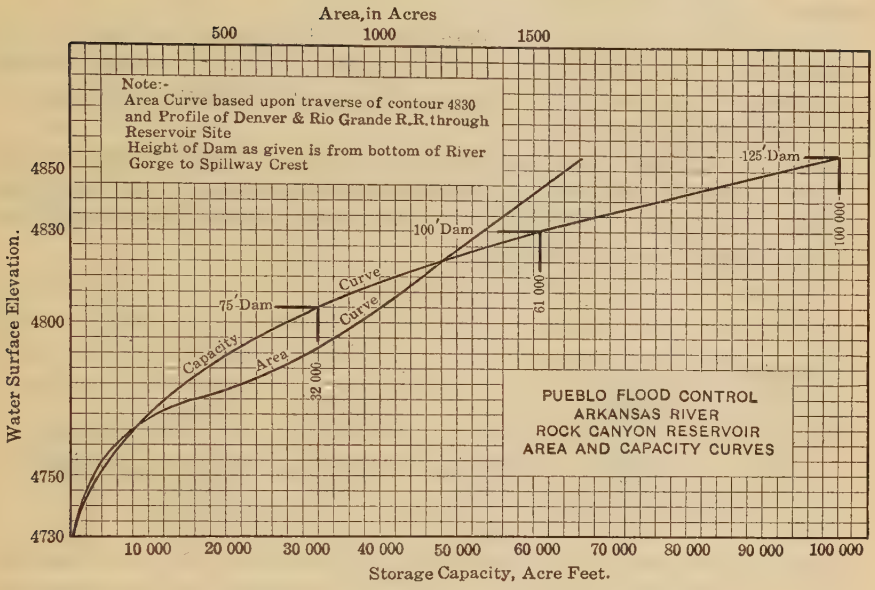

Area, in Acres
500
1000
1500
Note:-
Area Curve based upon traverse of contour 4830 and Profile of Denver & Rio Grande R.R. through Reservoir Site
Height of Dam as given is from bottom of River Gorge to Spillway Crest
125' Dam
100' Dam
75' Dam
Capacity Curve
Area Curve
100 000
61 000
32 000
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
ROCK CANYON RESERVOIR
AREA AND CAPACITY CURVES
Water Surface Elevation.
4850
4830
4800
4750
4730
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
Storage Capacity, Acre Feet.

FIG. 13.

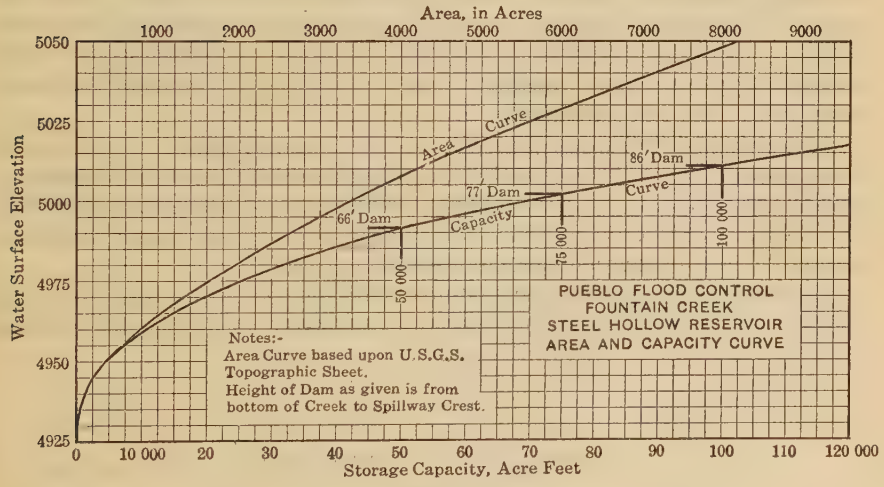

Area, in Acres
1000
2000
3000
4000
5000
6000
7000
8000
9000
Area Curve
86' Dam
77' Dam
66' Dam
Capacity Curve
100 000
75 000
50 000
PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
STEEL HOLLOW RESERVOIR
AREA AND CAPACITY CURVE
Notes:-
Area Curve based upon U.S.G.S. Topographic Sheet.
Height of Dam as given is from bottom of Creek to Spillway Crest.
Water Surface Elevation
5050
5025
5000
4975
4950
4925
0
10 000
20
30
40
50
60
70
80
90
100
110
120 000
Storage Capacity, Acre Feet

FIG. 14.

point on the bridge. This provision would probably necessitate raising the railroad grades generally throughout the city, as well as some of the street grades.

It should be noted that the enlargement of the channel through Pueblo, in order to carry the peak flow of the river, would not benefit property interests in the valley below; in fact, it might have the effect of slightly increasing the flood peaks in the lower river.

Plan C.—Another alternative plan for solving the flood problem on the Arkansas River is a combination of flood-detention storage and channel enlargement through Pueblo. Three combinations have been studied, resulting in the estimated costs given in Table 11.

TABLE 11.

Combination number.	Storage.		Channel Enlargement.		Total cost.
	Capacity, in acre-feet.	Cost.	Capacity, in second-feet.	Cost.	
1	32 000	$2 000 000	125 000	$4 500 000	$6 500 000
2	61 000	4 000 000	100 000	4 000 000	8 000 000
3	100 000	6 500 000	67 000	3 000 000	9 500 000

Other combinations can be readily selected from Fig. 19. It is believed that some combination like No. 1, with a comparatively small detention-storage reservoir and a large channel capacity, will prove to be the most economical.

Under Plan *C* the grade of all bridges should be raised in proportion to the channel capacity provided, and it is believed that a clearance of not less than 7½ ft. should be supplied between the high-water surface and the lowest point on the bridge, for channel capacities in excess of 50 000 sec-ft. This provision would necessitate raising the railroad grades, and some of the street grades, as in the case of Plan *B*.

It will be noted that the peak flow in the Arkansas River is quite materially reduced, even with the smallest detention reservoir considered (Combination No. 1), and that it is very considerably reduced with the larger reservoirs. Under Plan *C*, property interests in the valley below Pueblo would benefit in proportion to the detention storage provided.

Summary.—The resulting detention-storage channel capacities and estimated costs for flood control on the Arkansas River at Pueblo under Plans *A*, *B*, and *C* are summarized in Table 12.

TABLE 12.

Plan.	Detention storage, in acre-feet.	Channel capacity, in second-feet.	Estimated cost.
A	210 000	25 000	$17 500 000
B	None	168 000	5 500 000
C (Comb. No. 1)	32 000	125 000	6 500 000
C (Comb. No. 2)	61 000	100 000	8 000 000
C (Comb. No. 3)	100 000	67 000	9 500 000

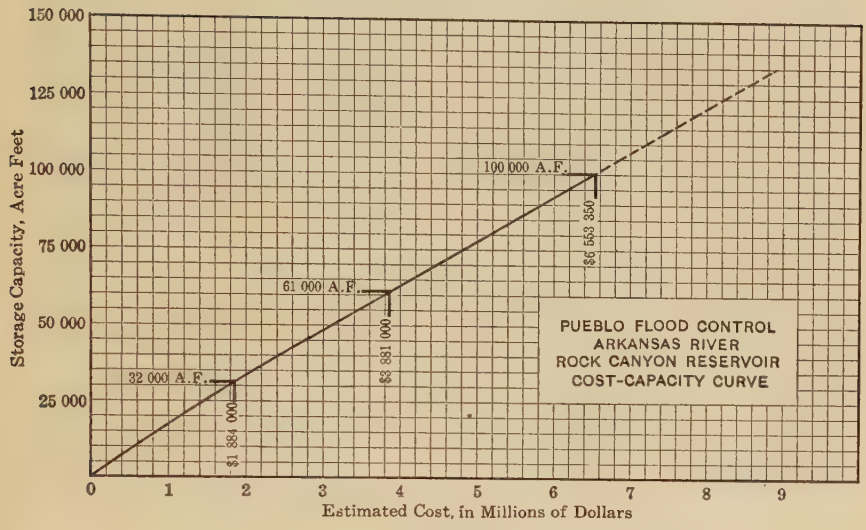

150 000
125 000
100 000
75 000
50 000
25 000
Storage Capacity, Acre Feet
100 000 A.F.
$6 553 350
61 000 A.F.
$3 831 000
32 000 A.F.
$1 384 000
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
ROCK CANYON RESERVOIR
COST-CAPACITY CURVE
0
1
2
3
4
5
6
7
8
9
Estimated Cost, in Millions of Dollars

FIG. 15.

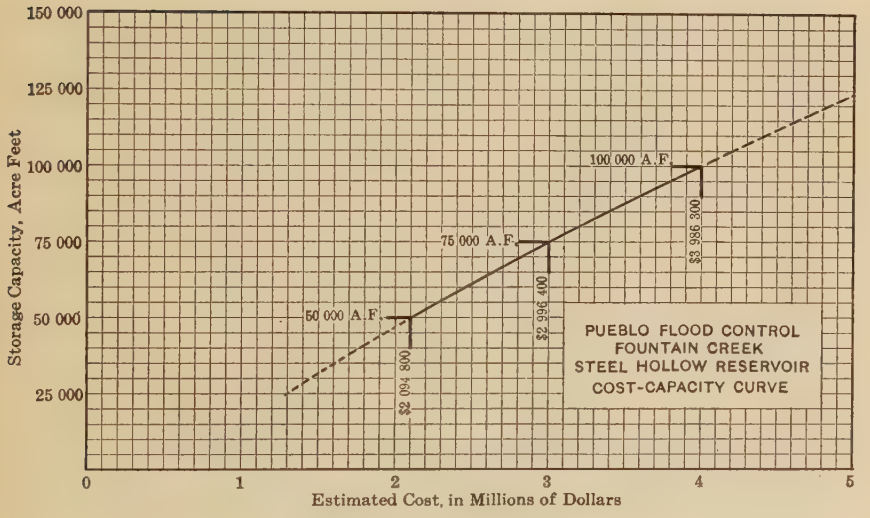

150 000
125 000
100 000
75 000
50 000
25 000
Storage Capacity, Acre Feet
100 000 A.F.
$3 986 300
75 000 A.F.
$2 996 400
50 000 A.F.
$2 004 800
PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
STEEL HOLLOW RESERVOIR
COST-CAPACITY CURVE
0
1
2
3
4
5
Estimated Cost, in Millions of Dollars

FIG. 16.

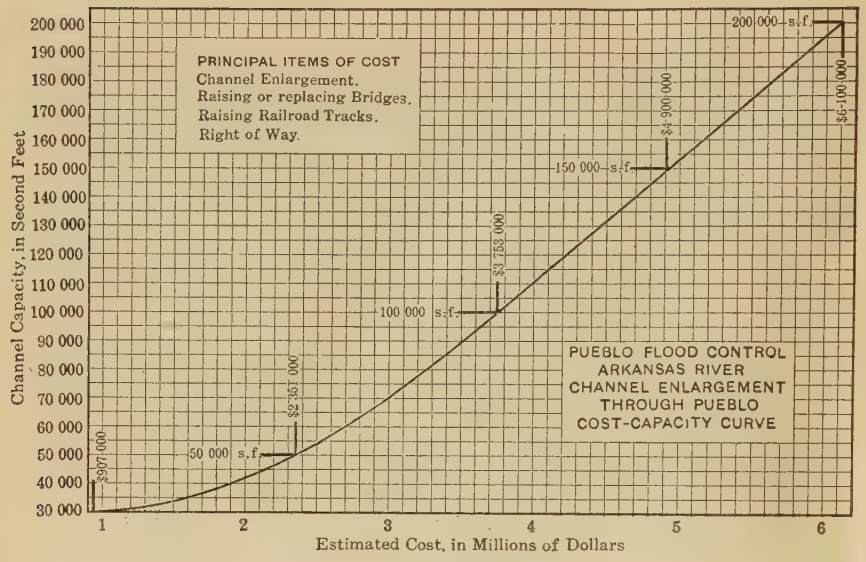

PRINCIPAL ITEMS OF COST
Channel Enlargement.
Raising or replacing Bridges.
Raising Railroad Tracks.
Right of Way.
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
CHANNEL ENLARGEMENT
THROUGH PUEBLO
COST-CAPACITY CURVE
Channel Capacity, in Second Feet
Estimated Cost, in Millions of Dollars
200 000
190 000
180 000
170 000
160 000
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
1
2
3
4
5
6
$907 000
50 000 s.f.
$2 351 000
100 000 s.f.
$3 753 000
150 000 s.f.
$4 900 000
200 000 s.f.
$6 100 000

FIG. 17.

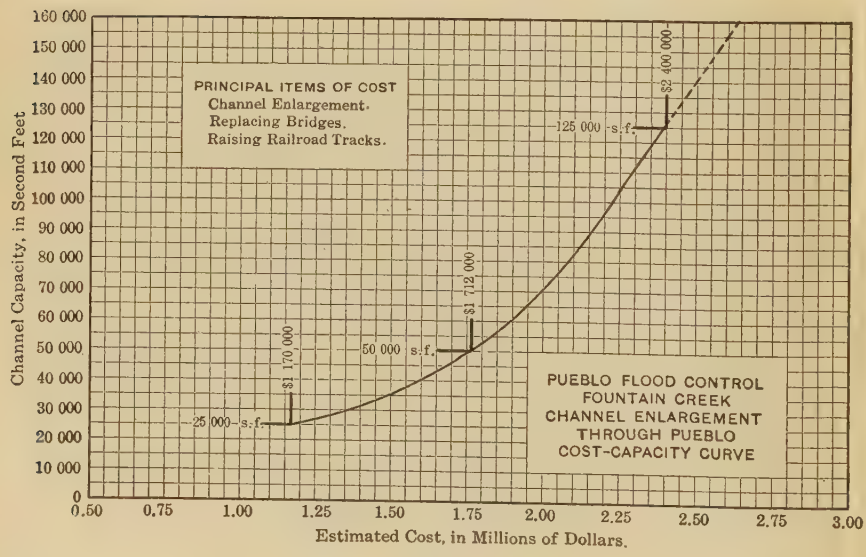

PRINCIPAL ITEMS OF COST
Channel Enlargement.
Replacing Bridges.
Raising Railroad Tracks.
PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
CHANNEL ENLARGEMENT
THROUGH PUEBLO
COST-CAPACITY CURVE
Channel Capacity, in Second Feet
Estimated Cost, in Millions of Dollars.
160 000
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
25 000 s.f.
$1 170 000
50 000 s.f.
$1 712 000
125 000 s.f.
$2 400 000

FIG. 18.

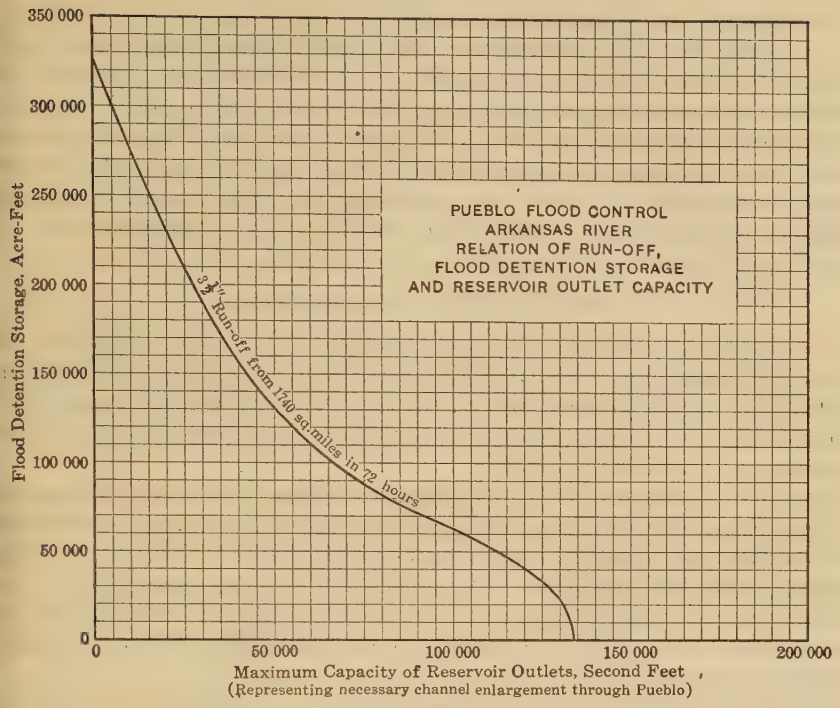

PUEBLO FLOOD CONTROL
ARKANSAS RIVER
RELATION OF RUN-OFF,
FLOOD DETENTION STORAGE
AND RESERVOIR OUTLET CAPACITY
3½" Run-off from 1540 sq.miles in 72 hours
Flood Detention Storage, Acre-Feet
350 000
300 000
250 000
200 000
150 000
100 000
50 000
0
0
50 000
100 000
150 000
200 000
Maximum Capacity of Reservoir Outlets, Second Feet
(Representing necessary channel enlargement through Pueblo)

FIG. 19.

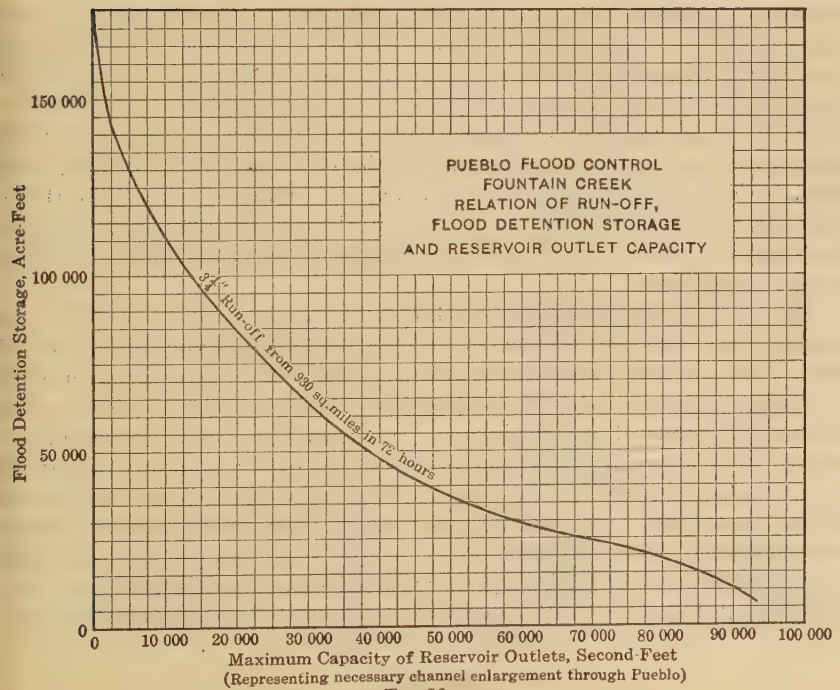

PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
RELATION OF RUN-OFF,
FLOOD DETENTION STORAGE
AND RESERVOIR OUTLET CAPACITY
3½" Run-off from 930 sq.miles in 72 hours
Flood Detention Storage, Acre-Feet
150 000
100 000
50 000
0
0
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
Maximum Capacity of Reservoir Outlets, Second-Feet
(Representing necessary channel enlargement through Pueblo)

FIG. 20.

Although Plan *B* appears to result in the lowest cost, it is believed that some combination of flood-detention storage and channel enlargement under Plan *C* may prove to be the wisest choice. This conclusion is based on the fact that property interests down the valley below Pueblo would benefit by the reduced peak flow under Plan *C*, and also on the belief that this plan may prove to be the most economical when final designs and estimates are prepared, particularly if a plan can be effected by which the railroad tracks would be left in their present location in the Rock Canyon Reservoir area.

Flood Control on Fountain Creek

In its present condition, Fountain Creek is a very serious menace to the City of Pueblo, due to the possibility that another flood might force the stream through the business part of the city, south of Mineral Palace Park. Four alternative plans for flood control on Fountain Creek have been considered and are discussed, as follows:

Plan D.—Under this plan it is proposed to control the floods by detention storage alone. Practically no data are available on which to base studies of possible flood-detention storage on Fountain Creek. However, a rough approximation has been made of a possible site 10 miles above Pueblo (the Steel Hollow Reservoir site) where the topographical sheet of the U. S. Geological Survey (scale, ½ in. = 1 mile; contour interval, 50 ft.) shows a basin of small capacity.

In the section of the paper dealing with drainage area and rainfall data (page 10), it has been estimated that a total volume of 164 000 acre-ft. and a peak flow of 110 000 sec-ft. would result from the maximum assumed storm on the Fountain Creek drainage area. The safe limit of flow in Fountain Creek under present conditions is estimated to be 25 000 sec-ft. From Fig. 20, it will be noted that the maximum assumed storm would be controlled by a detention storage reservoir with a capacity of 73 000 acre-ft., having an outlet capacity of 25 000 sec-ft., corresponding to the safe limit of flow in the present channel. The estimated cost of this amount of storage in the Steel Hollow Reservoir site is $3 000 000. Plan *D* would benefit property interests down the Arkansas Valley below Pueblo by reducing the peak flow in Fountain Creek to 25 000 sec-ft.

Plan E.—A solution of the flood problem on Fountain Creek by bank protection alone has been considered, and this plan appears to have considerable merit. The protective work would probably extend from a point north of the city limits to the junction with the Arkansas River, and would include protection to both banks of the flood-plain. In addition to bank protection, it is believed to be essential that all the bridge grades over Fountain Creek should be substantially raised. This would also necessitate raising the railroad grades in some instances. Preliminary studies indicate that the most effective and economical protection may result from a slope paving of hot, dumped slag, protected at the toe of the slope by interlocking steel sheet-piling. It is believed that Fountain Creek could be made entirely safe by bank protection

of this kind. Plan *E*, of course, would not benefit property interests in the Arkansas Valley below Pueblo, inasmuch as it would have no effect in reducing the flood peaks from Fountain Creek.

Plan F.—Consideration has been given to the possibility of controlling the floods of Fountain Creek by confining the stream in a concrete-lined channel for a reach of about three miles above its junction with the Arkansas River. Plan *F* would also involve raising bridge and railroad grades, as in the case of Plan *E*. Rough estimates show that this plan would cost approximately $2 500 000 for a channel of sufficient capacity to carry the estimated peak flow of 110 000 sec-ft. This plan, it is believed, will warrant careful consideration. It should be noted that Plan *F* would not benefit property interests in the Arkansas Valley below Pueblo.

Plan G.—Another alternative plan for flood control on Fountain Creek is embraced in a combination of flood storage and either bank protection or channel enlargement. Two combinations of detention storage and channel enlargement have been considered, resulting in the costs given in Table 13.

TABLE 13.

Combination number.	Storage.		Channel Enlargement.		Total cost.
	Capacity, in acre-feet.	Cost.	Capacity, in second-feet.	Cost.	
1	25 000	$1 500 000	68 000	$2 000 000	$3 500 000
2	50 000	2 000 000	38 000	1 500 000	3 500 000

Under Plan *G*, the bridge and railroad grades would be raised in proportion to the channel capacity provided. It will be noted that the peak flow in Fountain Creek is quite materially reduced, even with a small detention reservoir. A comparison of the costs under this plan with the cost under Plan *E* or Plan *F* suggests the probability that bank protection alone or channel enlargement alone will prove to be more economical than any combination with detention storage. Property interests in the Arkansas Valley below Pueblo would be benefited in proportion to the reduction of the peak flow in Fountain Creek under Plan *G*.

Summary.—The resulting detention-storage channel capacities and estimated costs for flood control on Fountain Creek under Plans *D*, *E*, *F*, and *G*, are summarized in Table 14.

TABLE 14.

Plan.	Detention storage, in acre-feet.	Channel capacity, in second-feet.	Estimated cost.
D	73 000	25 000	$3 000 000
E	None	110 000	2 500 000
F	None	110 000	2 500 000
G (Comb. No. 1)	25 000	68 000	3 500 000
G (Comb. No. 2)	50 000	38 000	3 500 000

Either Plan *D* or Plan *G* would provide considerable protection against flood damage to property interests down the Arkansas River below Pueblo, and the additional cost for this protection might be warranted. However, the estimated costs of these alternative plans are so close and the data on which they are based are so meager that definite conclusions are impossible as to the most attractive plan.

Irrigation Storage and Power Development

Preliminary consideration has been given to the possibility of utilizing a part of the storage capacity in the Rock Canyon Reservoir site or of providing other storage reservoirs for irrigation use. Some consideration has also been given to the possibility of developing a limited amount of power at the Rock Canyon Reservoir site for municipal purposes and utilizing some of the storage capacity in this reservoir for the domestic water supply of Pueblo.

The total acreage under irrigation in the Arkansas Valley in Colorado is about 450 000 acres. Complete records for sixteen years indicate that the average total run-off of the Arkansas River at Pueblo is 533 000 acre-ft. Rainfall records indicate an average rainfall of 0.86 ft. during the irrigation season from March to November, amounting to 388 000 acre-ft. over the irrigated area of 450 000 acres. The total average rainfall plus run-off at Pueblo is, therefore, 921 000 acre-ft., or only slightly more than 2 acre-ft. per acre, not allowing anything for canal losses by seepage and evaporation, which losses are known to be large. When it is considered that these figures are averages and that there are a great many years when the run-off is materially less, it is evident that the supply for additional irrigation development is very limited. If, however, ample storage capacity was available on the river above Pueblo at reasonable cost, it might prove feasible to develop hold-over storage in sufficient quantity to be of some value in augmenting the supply to present canals or in furnishing a supply to new acreage. This possibility should be given further study when full information is available, notwithstanding the fact that the data at hand indicate no feasible project.

It would not be possible, of course, to utilize any of the flood-detention storage capacity for irrigation storage, and any capacity reserved for irrigation use would have to be additional to the required detention storage. Any capacity developed for irrigation storage in conjunction with a detention reservoir would be at the bottom of the reservoir, and the permanent outlets for detention storage would be placed at the top of the irrigation storage. Irrigation storage located below the permanent outlets would be subject to comparatively rapid silting up and, in this connection, it is believed that the yearly loss by silting might amount to as much as 0.25% of the total average run-off, or about $0.25 \times 533\,000 = 1\,330$ acre-ft. per year. This loss by silting should be given consideration, particularly in the case of a small development for irrigation storage.

The possibility of utilizing a small amount of storage capacity in the Rock Canyon Reservoir site as a domestic supply for the City of Pueblo has been considered, but this plan does not appear to be feasible, due to the fact

that any small reservoir on the main channel of a silt-laden stream would soon be filled with silt. It is believed that the settling reservoir would have to be located off the main stream and fed by a canal similar to the plan now followed. Such an arrangement does not appear to be feasible in connection with the Rock Canyon Reservoir.

The possibility of developing power at the Rock Canyon Reservoir site would depend on whether or not irrigation storage was developed. If no irrigation storage was developed, and the permanent outlets of the detention reservoir were placed at the lowest point in the reservoir, as would be the case, there would be no head for power development. If irrigation storage was developed in any considerable quantity, there might be a possibility of developing a limited amount of power. However, it is believed that very little, if any, firm power would result from such an installation, due to the fact that the development of any considerable irrigation storage would involve holding the water over, possibly for several years at a time.

DISCUSSION

ARTHUR O. RIDGWAY,* M. AM. SOC. C. E. (by letter).—For the sake of clarity in discussing this admirable paper, any comment should conform in sequence to the different sections outlined therein.

History of Former Floods.—A thorough study of all available sources of information indicates that no flood of the magnitude of the 1921 deluge had occurred at the site of Pueblo for at least 100 years previously. Indians and early settlers left stories of a great flood in 1844, but, on investigation, this flood was found to be undoubtedly the result of a tremendous snow which fell in the valley in the early spring and did not entirely disappear for several weeks afterward. The evidence collected shows that although probably of greater volume, because of vastly longer duration, it could not have exceeded in height the flood of 1864. Evidence is also at hand to the effect that, for many years prior to 1844, no such flood as that of June, 1921, could have occurred. That the flood of 1894 substantially exceeded the one which occurred in 1864 has been definitely determined, and that the recent flood of June, 1921, was far greater than the 1894 deluge is not open to question.

In connection with these several floods, the evolution of the channel of the Arkansas River through the site of Pueblo is most interesting and effects a valuable contribution to the information necessary in planning preventative or protective measures. The authors state that at the time of the flood of 1894, "Pueblo had little or no river protection, and the Arkansas River meandered through the city, cutting its banks and changing its course." This is not strictly correct.

Definite data are at hand, which show the various channels of the Arkansas from 1872 to the present time. It is more than probable that the position of the channel in 1872 reflects conditions extant during the flood of 1864. This channel had a greater width than any subsequent channels through the main part of the town, and notwithstanding the fact that the course was very sinuous, its carrying capacity was no doubt quite comparable with that of later channels.

Some time between 1872 and 1881, whether by encroachment of the growing city on the older channel, or by actual channel improvement work, as to which need not now be determined, the channel through the town underwent a decided straightening and narrowing. Its position in the lower part of the city was entirely changed, eliminating a serious detour which existed in the older channel. Between 1881 and 1889, the straightening continued to some extent. The floods of 1889 gave an impetus to channel improvement work, and, by 1894, the channel was practically confined through the built-up part of the city. At the time of the flood of 1894, additional levee work was in progress, especially in the vicinity of the West Fourth Street Viaduct. Subsequently, the levees were completed through the city, the bridges were raised, and the channel otherwise was improved to carry a flood equal in magnitude to that which occurred in that year, the volume of which was determined at the time to be about 40 000 sec-ft. The programme of these improvements was not

* Asst. Chf. Engr., Denver and Rio Grande Railroad, Denver, Colo.

entirely completed until about 1898, since which time little or no additional protective work has been effected. Since the completion of the channel improvements, floods of considerable magnitude have occurred at times, but on no occasion has there been any serious damage and apparently no serious menace. The flood of June, 1921, so far exceeding any of which there is a recorded history, was totally unexpected and no doubt will be included in the list of great American disasters.

Description of June, 1921, Flood.—The Arkansas River at Pueblo quickly receded from flood crest on the night of June 2d, 1921, to nearly normal stage by 8.00 A. M., June 3d, without leaving any ill effects on the channel, and remained at that stage until 5.00 P. M., of June 3d; therefore, only the events which transpired subsequent to that time are involved.

The Deputy State Engineer has fixed the time of overflowing of the channel at the Main Street Bridge, where the river gauge is located, at 8.45 P. M., June 3d, and has computed the discharge of the stream at that time to have been 45 000 sec-ft., with a gauge height of 18.14 ft. The floor of the Union Depot has an elevation equivalent to a gauge reading of 17.61 ft., or 0.53 ft. lower than the coping of the channel walls at the Main Street Bridge. Throughout the night of June 3d, a log was kept in the Union Depot of the various stages of the water, both during its rise and recession. This log, perhaps the only one in existence compiled from actual measurements co-ordinated with accurate and frequent time intervals, shows the stages and duration of the overflow peak, as given in Table 15.

TABLE 15.—FLOOD STAGES AT UNION DEPOT, PUEBLO, COLO., JUNE 3D AND 4TH, 1921.

Time of rise (read up), June 3d.	Gauge height extended, in feet.*	Time of fall (read down), June 4th.
11.55 P. M.	27.36	12.01 A. M.
11.54 P. M.	27.00	12.08 A. M.
11.46 P. M.	26.00	1.03 A. M.
11.41 P. M.	25.00	1.22 A. M.
11.36 P. M.	24.00	1.37 A. M.
11.31 P. M.	23.00	2.02 A. M.
11.10 P. M.	22.00	2.28 A. M.
10.43 P. M.	21.00	3.01 A. M.
10.12 P. M.	20.00	3.47 A. M.
9.43 P. M.	19.00	5.02 A. M.
9.05 P. M.	18.00	7.15 A. M.

* Floor of Union Depot = gauge height of 17.61 ft.

The figures in Table 15 clearly indicate the remarkably short duration of the overflow and the very rapid rise of the river, especially between the hours of 11.31 and 11.46 P. M., when the rate was 1 ft. for each 5-min. period. The subsidence, although phenomenally rapid, was slightly slower. The peak occurred at 11.55 P. M., and not at 1.00 or 1.30 A. M., as heretofore assumed. At no time during the period of either rise or fall does the stage appear to have remained stationary; even the highest stage was only momentary. In view of the position of the Union Depot with respect to the river channel, and considering all the other relevant circumstances, there seems to be no

sound reason for not considering the rise as actually representing conditions on the Main Street gauge. The subsidence at the Union Depot, although in all probability lagging slightly, should be regarded as exactly parallel with that of the channel. The crest at the Union Depot reached a gauge of 27.36 ft., as compared with the reported peak gauge of 24.66 ft., taken from a high-water mark near the Main Street Bridge. Reference of the Union Depot readings, observed in comparatively quiescent water during the progress of the overflow, to the gauge datum has been confirmed by spirit leveling, and there seems to be no question as to their accuracy.

There is little doubt but that the river had fallen to the channel overflow point by about daylight. This is confirmed in a general way by the log at the Union Depot, for with the more than 1 ft. of mud deposited on the depot floor, the observer could scarcely determine just when the water left the structure. In determining the volume of the flood, the important fact for consideration is that notwithstanding the river had fallen to the overflow point at the Main Street Bridge, substantial volumes were flowing, with no insignificant velocities, through other parts of the city from the broken levees near the West Fourth Street Viaduct.

The flood in the Fountain River can now be said with a considerable degree of certainty to have joined the Arkansas River on the eastern outskirts of Pueblo at about 3.00 A. M., on June 4th. This was 3 hours after the peak was reached by the Arkansas, or when that stream had fallen to a stage within 3 ft. of the overflow height. The discharge of the main stream, therefore, could not have been more than 60 or 70% of the peak flow, whatever that was, when it was joined by the Fountain River which from an accurate flood section is now estimated to have carried a maximum of about 40 000 or 45 000 sec-ft.

In a supplemental report, R. G. Hosea, Deputy State Engineer, gives some results of his field investigation of the tributaries of the Arkansas River east of Pueblo, together with a computed rate of discharge of the main stream near La Junta, Colo., 65 miles east of Pueblo. These results are as follows:

St. Charles	50 000 sec-ft.
Chico	20 000 " "
Huerfano	5 000 " "
Arkansas at La Junta	180 000 to 200 000 " "

As flood crests in the tributaries were not synchronous with the main stream, nor with each other, it is more than probable that the maximum discharge of the river at La Junta did not exceed 200 000 sec-ft.

At 9.30 A. M., on June 5th, the Schaeffer Dam, an earthen structure, 90 ft. high, on Beaver Creek and about 11 miles from its junction with the Arkansas, failed with a crash. The impounding reservoir, estimated to contain about 4 000 acre-ft. at the time of failure, was emptied in 30 min. The attendant flood reached Swallows, a point on the river 8 miles below the mouth of Beaver Creek, at 11.30 A. M.; Goodnight, 10 miles farther down, at 1.30 P. M.; and Pueblo, an additional 5 miles, at 2.15 P. M. The itinerary of this wave may be scheduled as shown in Table 16.

TABLE 16.

Place.	Flood arrived.	Lapsed time.	Distance traveled.
Reservoir site	9.30 A. M.	.....	.....
Swallows	11.30 A. M.	2 hours	19 miles
Goodnight	1.30 P. M.	2 hours	10 "
Pueblo	2.15 P. M.	0 hours 45 min.	4 "
Total	.....	4 hours 45 min.	33 miles

It is important to note that, at its crest, the flood, as recorded at the Main Street gauge, did not exceed 28 000 sec-ft. at a gauge height of 13 ft., or more than 5 ft. below the 18.14-ft. gauge of overflow point. In all probability if there had been no breaks in the levees from the flood of June 3d, the channel would have carried the flow from the Schaeffer Reservoir. The most important fact, however, is that notwithstanding the reading on the Main Street gauge was 5 ft. below the overflow point, the water was several inches above the floor of the Union Depot, and probably reached a gauge height there of about 18.00 ft., the overflow point at Main Street. This is direct confirmation of a previously mentioned observation to the effect that when the river had fallen to its channel overflow point at Main Street, in the early morning of June 4th, with a rated discharge flow of 45 000 sec-ft., large volumes of water were flowing through other parts of the city from breaks in the levees near the West Fourth Street Viaduct. Although the total volume from the Schaeffer Reservoir was relatively small and effected no serious damage at Pueblo and points farther east, yet, due to the release in the body, the damage inflicted in the valley west of Pueblo was far greater than in the previous flood, and left a real "valley desolate".

Flood Loss.—The authors are to be congratulated in assembling even preliminary figures on this most illusive factor of property loss, and there seems to be no warrant for suggesting change in the amount submitted. The figures are of inestimable value in planning protective or preventative measures and are an index of the justifiable extent to which such measures can be recommended. Doubtless, these amounts, together with the possibility of a future occurrence of a flood of equal or greater magnitude, will be weighed against the cost of assured protection. It is too bad, though, that mention was not made of the most vital factor in the whole matter. More than 150 lives were lost—the exact number can never be known. Thus far, 78 bodies have been recovered. Men might ponder at great length as to the justifiable investment to guard against property loss, but if the people in this part of the Arkansas Valley must return to their industrial pursuits, and it is assumed that they must, there is no warrant for hesitancy. The futility of placing monetary value on a human life should not restrain the engineer as a true conserver from placing personal safety in industry far above all things else.

Estimated Peak Flow and Volume.—The peak flood of the Arkansas River through the City of Pueblo was in all probability about 100 000 sec-ft., as first estimated by Mr. Hosea. This figure will be confirmed or corrected when the investigations now in progress are completed. The channel had a flood capacity of about 40 000 sec-ft., without any margin of safety. Improvements necessary to insure such capacity with a reasonable degree of safety could be effected at nominal expense. It would seem, therefore, that, in considering protective measures involving substantial investment, provision is necessary only for those floods of a greater discharge rate than 40 000 sec-ft. The volume in the Arkansas, passing through the city in excess of a discharge rate of 40 000 sec-ft. could not have been more than 30 000 acre-ft. The Fountain River, flooding a safe channel with a capacity of 25 000 sec-ft., for 12 hours, with a peak discharge of 45 000 sec-ft., would have a volume of about 10 000 acre-ft. without channel provision. In other words, if 30 000 acre-ft. had been held back on the Arkansas for a few hours and 10 000 acre-ft. on the Fountain, no disaster would have occurred at Pueblo on the night of June 3d.

It does not appear that the combined maximum flow of the Arkansas and Fountain of 65 000 sec-ft., together with the accretions of the tributaries east of Pueblo, would work any hardship on the lower valley. Prior to the June flood, a peak flow greater than 40 000 sec-ft. had not passed Pueblo in the Arkansas for a period of 100 years or more, if historical deductions are correct. The average daily discharge for a period of 11 years—1910-20—was 770 sec-ft.

TABLE 17.—INCHES OF RAINFALL, JUNE 2D-6TH, 1921.
(All Gauges Read at 6 P. M., Except as Noted.)

Station.	Drainage basin.	Day.					Total. 2d-6th.
		2d.	3d.	4th.	5th.	6th.	
Monument	Fountain	....	.....	2.90	0.82	0.05	3.77
Colorado Springs	"	....	0.50	4.40	1.26	0.42	6.58
Fremont	"	....	2.53	2.61	1.43	0.48	7.05
Lake Moraine	"	....	0.65	3.68	1.40	0.18	5.91
Victor*	Oil Creek	....	0.03	2.08	1.55	0.37	4.03
Canon City†	Arkansas	....	0.30	2.35	0.75	0.40	3.80
Florence†	"	....	0.99	3.31	2.47	0.13	6.90
Penrose†	Beaver	....	.....	7.00	2.00	1.50	10.50
Pueblo‡	Arkansas	1.94	1.64	1.45	1.12	0.09	6.15
Buena Vista	"	....	.....	0.90	0.42	....	1.32
Leadville	"	....	.....	0.16	0.49	0.71	1.36
St. Elmo	Chalk Creek	....	.....	0.63	0.55	0.38	1.56
Hayes Ranch	Osteen Creek	....	10.00	Friday evening and night.			10.00
Hobsin Ranch	Arkansas	....	6.50	"	"	"	6.50
Teller	Turkey Creek	....	7.50	"	"	"	7.50
Boggs Flat	Boggs Creek	....	14.00	"	"	"	14.00
Higgins	Eight-Mile	....	12.00	"	"	"	12.00

* Gauge read at 4.00 P. M.
† " " " 8.00 A. M.
‡ " " " midnight.

Drainage Area and Run-Off Data.—It is true that the part of the Arkansas Basin between Canon City and Pueblo supplied nearly all the run-off in the flood of June 3d, yet by no means can the drainage basin of 3 047 sq. miles

above Canon City be ignored in considering protective measures. Notwithstanding the fact that the greatest discharge at the Hanging Bridge in the Royal Gorge since the construction of the railroad in 1880 has probably not exceeded 9 000 sec-ft., Grape Creek, which flows into the Arkansas just below the Grand Canyon, has a formidable drainage area of 483 sq. miles and is subject to excessive floods. The railroad branch in the canyon occupied by this stream, extending from Canon City to Silver Cliff, was several times seriously damaged by floods, and, in 1889, after only nine years of operation, was so nearly destroyed that it was permanently abandoned.

Table 17 shows the daily and total precipitation measured by various observers at points in the Fountain and Arkansas Basins during the storms of June 2d-6th, 1921. Attention is directed to a previous error in the Colorado Springs precipitation for June 3d which is corrected herein.

It is inconceivable that the heavy precipitation of Friday evening and night could extend over any considerable area, and equally irrational to apply the averages deduced therefrom over the entire drainage basin of either the Fountain or Lower Arkansas Rivers. An incontrovertible fact of long standing is that by far the greater volume of precipitation in the Colorado Arkansas is the result of violent rain storms. There should be no hesitancy in saying that the basin, especially the lower part, is infested with cloudbursts, for no other nomenclature better fits the observed phenomena. Grouped according to elevation, these stations furnish the results shown in Table 18.

TABLE 18.—AVERAGE INCHES OF RAINFALL, STATIONS ABOVE PUEBLO, COLO., IN 72 HOURS, JUNE 2D-6TH, 1921.

Elevation, in feet.	Number of stations.	Average inches of rainfall, 72 hours.
4 600 to 5 000	6	9.17
5 000 to 6 000	4	6.94
6 000 to 7 000	1	3.77
Above 7 000	6	3.49
Total.....	17	6.41

It will be observed from Table 18 that, for this particular series of storms, the precipitation varies inversely with the altitude, a fact contrary to the commonly accepted law which doubtless will apply only to precipitation from general storms. Even in the mean annual precipitation, there is a slight vagary in this law in the Upper Arkansas Basin. No reason is advanced as to why the probable maximum precipitation in 72 hours should be exactly 140%, rather than any other multiple of the greatest precipitation of record; neither is it clear that the figures for percentage of run-off will apply to this particular region. In the 11 years, 1910-20, the ratio of mean annual run-off to mean annual precipitation for the basin above Canon City was 23%, and 19% for the entire basin above Pueblo. On the other hand, it is easy to conceive that the ratio from cloudbursts, in some of the tributaries with exces-

sive gradients, might closely approach 100 per cent. Apparently, the law of averages will not apply in this case, except in a very limited manner; rather should the analysis consist in a study of each individual tributary. For example, it is known that Oil Creek, with a drainage area of 416 sq. miles, never floods frequently or excessively, while, in contrast, Rock and Peck Creeks are subject to frequent floods. Hardscrabble has idiosyncrasies well worthy of analysis. Other tributaries have similarly developed certain characteristics, the permanency of which has not yet been assailed. Doubtless, the position of the confining mountain ranges, their altitude, and their relation to each other, as well as to the Great Plains, have a great deal to do with the case.

It may be possible to construct from past records, combined with personal experiences and accumulated data, a synthetic programme of a probable maximum flood at Pueblo. The apparently insurmountable obstacle in this programme will be the occurrence and intensity of cloudbursts, the forecasting of which has thus far baffled all attempts at solution.

Flood Control.—The answer to the problem seems to lie in some scheme of flood-regulating reservoirs. This is by far the safest plan, for, regardless of the magnitude of the flood peak, discharge through the channel will be limited to its capacity. The only further concern will be in ascertaining the volume of maximum flood which it will be necessary to hold back, and this factor is more susceptible of definite determination than the peak flow of any imaginary flood.

Obviously, the channel through the city should be as large as ultimate economy will permit. The larger the city channel, the smaller will be the required detention. This is a distinct advantage in that it will reduce the cost of maintaining reservoir capacity against the deposit of enormous quantities of débris brought down by the torrential flood velocities of the river and its tributaries. On the other hand, the city channel ought to be as small as possible, for, otherwise, property suited for intensified use would be devoted to only occasional service during extraordinary floods. An infrequently used flood channel, many times larger than would be required for normal flow, would constitute waste land in the heart of the city and impose extra burdens on the daily conduct of city affairs. Again, a city channel of maximum flood capacity would be no safeguard to life and property in the valley east of Pueblo, whereas confinement of the river within minimum and permanent limits would permit the cultivation of land otherwise unproductive.

In the interest of conservation, it would seem, therefore, to be necessary first, to determine the size of the channel which the value of property both urban and rural can consistently bear, and weigh this cost with that of reservoirs to supplement the channel in passing the floods. The only unknown quantity in the problem is the volume and peak flow of the possible maximum flood. The suggestion of a rainfall 40% in excess of the two greatest storms on record, distributed over the water-shed according to altitude and running off at assumed rates, is not entirely satisfactory. There is no certainty that the records of the 1894 flood tell the whole story, and it is quite certain that the records used for the 1921 flood do not. Moreover, the altitude distribution

over the whole water-shed of the few available averages and the assumed percentages of run-off do not seem wholly applicable to the Arkansas Basin. Here, the precipitation is spotted, storms are greatly concentrated in area, flood rainfall is extreme in intensity, and run-off is widely variant.

As the concern in the maximum peak discharge is only for the purpose of providing requisite channel capacity without control reservoirs, it may be that the suggestion of 168 000 sec-ft., as crest discharge in the Arkansas, will answer the purpose. However, it is entirely possible, with the same intensity of storms and slightly different distribution over the tributary basins, to have greater peak flows with smaller volumes than even that which occurred on June 3d, 1921. In this phase of the problem, the vital factor is the torrential rains in certain tributaries, which might make their peak discharges coincident at the critical point in the river. The difficulty lies in the local concentration of storms in places where no long-time records have been kept. Similarly, it may be that the figures of 322 000 acre-ft. in the Arkansas River represent the greatest possible volume of flood of a few days' duration, but as the method by which they were deduced is open to question, their reliability is not assured.

As previously mentioned, the present channel of the Arkansas River can and did carry 40 000 sec-ft., and can be made to do so with an adequate margin of safety at nominal expense. Apparently, therefore, the detention reservoir capacity should be computed on this basis. The development of the Rock Canyon site to requisite capacity, even with this promise, may not be sound economy, but it does not yet appear to be impracticable to obtain a part of the detention capacity on some of the tributaries. For instance, Dry, Boggs, and Rock Creeks were the worst offenders in the recent flood. Peck Creek is often and Hardscrabble occasionally the culprit, but this time neither was very unruly. The flood duration on all these streams is very brief. From data collected in the field, the maximum rate of discharge and total volume of flood for the three first-named streams may be approximated as follows:

	Second-feet.	Acre-feet.
Dry Creek	17 000	7 000
Boggs Creek	33 000	13 500
Rock Creek	36 500	15 000

The channels of these streams and the river below their conjunction therewith will carry substantial flows without damage, and apparently reservoirs of adequate capacity for their control would be quite practicable. On investigation other tributaries will doubtless yield even greater promise of advantage.

It is entirely practicable, although somewhat expensive, to remove the railroad tracks and the Bessemer Canal from any reservoir basin at the Rock Creek site. The suggestion of leaving these works within the basin to be subject to occasional submergence as an economic measure is not at all clear. Aside from that incurred by the failure of the Schaeffer Dam, most of the railroad damage west of Pueblo accrued from floods in the tributaries. Any advantage gained by checking main stream velocities within the reservoir limits would doubtless be trifling in contrast to the enormous loss from inundation and subsequent excessive cost of clearing the submerged works from a heavy deposit of detritus.

The topography of the Fountain River is not as favorable as that of the Arkansas for the construction of retarding reservoirs, but the channel through the town is more susceptible of improvement and enlargement. Any reservoir site on the Lower Fountain River, and there seems to be none of any great promise, will be expensive for the installation of control works, and, perhaps, prohibitive in cost per acre-foot capacity. It is also quite likely that the difficulty of securing satisfactory foundations will militate against the adoption of a detention plan.

A consideration of all the phases of the situation indicates preference for the improvement and enlargement of the channel through the city as the best plan. This view is influenced by the probability that after a thorough investigation the peak flow of any future maximum flood will be taken at substantially less than the suggested 110 000 sec-ft. Present information indicates that the probable maximum crest will exceed the 45 000 sec-ft. of June 4th, 1921, by only a small proportion. However, any plan for flood control on the Fountain River must be co-ordinated with plans for controlling the Arkansas River, in order to obtain the desired results in the valley east of Pueblo.

The authors are to be commended most highly for including even a roughly approximate statement of cost for the various plans suggested. This information will doubtless serve the desired purpose of emphasizing the gravity of the entire situation. With still only very meager data at hand, it is impossible to confirm or revise the estimates. Attention is called to a possibility of reducing the amounts very materially after a more mature investigation as to whether the Fountain peak of 45 000 sec-ft. of June 3d and 4th, 1921, was not a probable maximum, or nearly so, and also by basing the Arkansas computations on a channel capacity of 40 000 sec-ft.

The utilization of water accumulated by detention reservoirs for either irrigation or power purposes does not now appear to be entirely practicable. Detention reservoirs must be emptied as speedily as the channel capacity will permit, for in some seasons torrential rains in the water-shed have a pernicious habit of rapid repetition. The storage of large quantities of water to be used slowly could only be accomplished by quickly drawing off the detained volumes into other basins through canals of large dimensions. Still, this feature of the case is a move toward the utmost utilization of natural resources, and it is worthy of some consideration and investigation.

R. G. Hosea,* Esq. (by letter).—Since the Arkansas River flood of June 3d, 1921, two questions frequently asked of the State Engineer's Office, are:

1. What was the peak flow of the flood at Pueblo?

2. What quantity could have been safely carried through Pueblo, in the existing river channel?

The first question is extremely difficult to answer and probably may be estimated more intelligently after a consideration of the second.

Through the main part of Pueblo, the channel of the Arkansas River is approximately 150 ft. wide, between nearly vertical masonry walls 18 ft. high. The State gauging station was located at the Main Street Bridge, where frequent measurements were made at low-water stages by wading and from

* Deputy State Engr., Denver, Colo.

the bridge during high water. On June 2d, at 10.00 A. M., the following measurement was made by Messrs. H. D. Amsley and G. C. Price, State Hydrographers: Gauge height, 4.69 ft.; area, 444 sq. ft.; discharge, 2 417 sec-ft.; and mean velocity, 5.44 ft. per sec.

From 6.00 to 6.20 P. M., on June 3d, these men also made a float measurement by timing with stop-watches logs and drift over a 100-ft. course. This measurement is only a rough approximation, since the river rose 1 ft. (from Gauge 10.6 to Gauge 11.6) during this time, and it was impossible to obtain accurate soundings. The depths, therefore, were computed, using 11.2 ft. as the mean gauge height for the period. The velocities thus obtained were 10 ft. per sec. near the side-walls and 15.2 ft. per sec. in the central part of the channel.

The area computed, using 11.2 ft. as the gauge height (and making an assumption as to the amount of scouring of the channel, as explained later), was 1 505 sq. ft. The mean velocity was 11.8 ft. per sec., based on the following assumptions: Mean velocity = 0.90 of surface velocity; surface velocity for 30 ft. from each wall = 10 ft. per sec.; and surface velocity for 90 ft., central part of channel = 15.2 ft. per sec. This gives a discharge of 17 905 sec-ft.

The channel at the Pueblo Station is subject to scouring during high water and filling again under normal conditions. This condition was particularly apparent before and after the flood of June 3d. A measurement made on June 7th showed: Gauge height, 5.60 ft.; area, 788 sq. ft.; discharge, 6 274 sec-ft.; and mean velocity, 8.0 ft. per sec.

Comparing this measurement with that of June 2d, it is seen that the gauge height was 0.91 ft. higher, corresponding to an increase in area of 0.91×150, or 136.5 sq. ft., while the measured area showed 344 sq. ft. more, leaving an excess difference of 208 sq. ft. caused by scouring. This amounts to an average increase in depth of 1.4 ft. in a channel 150 ft. wide (in one place, the actual increase was slightly more than 3 ft.). In computing the area for the float measurement of June 3d, it was assumed that one-third of this scouring had taken place. A measurement on June 29th, showed: Gauge height, 4.77 ft.; area, 542 sq. ft.; discharge, 2 865 sec-ft.; and mean velocity, 5.38 ft. per sec. This area when compared with that of June 7th, shows that during this time the channel filled an average of 0.8 ft. over the 150-ft. section, and was evidently approaching its condition prior to the flood.

The Main Street Bridge at Pueblo is an old-fashioned Bollman truss, with floor-beams, spaced about 10 ft. apart, extending below the lower chord. Levels taken a few days after the flood showed the following gauge heights: Bench-mark on stone coping (top of channel side-wall), Elevation 4875.19, equals 18.18 ft.; sidewalk of bridge, 18.66 ft.; top of floor-beams, 17.68 ft.; bottom of floor-beams, 14.68 ft.; and bottom of stringers, 16.85 ft. The high-water mark on the City Auditorium, adjacent to the bridge, is 4 681.71, or 6.5 ft. above the top of the channel coping.

It is evident that the channel could not safely carry water over a gauge height of 14.68 ft. and at this elevation it would reach the bottom of the floor-beams. However, as an estimate of what might have been carried in the channel, a gauge height of 16.85 ft. was used as the clearance line of the bottom of

the lower stringers of the bridge. Probably the most accurate determination of the flow at this gauge height may be made from a logarithmic discharge curve, based on previous measurements and on the measurements already referred to, since this will be a straight line and can be accurately extended.

At a gauge height of 16.85 ft., such a curve gives a discharge of 45 000 sec-ft. From this, and the computed area (approximately 2 300 sq. ft.), a velocity of 19.5 ft. per sec. is obtained. Extending a velocity curve is not satisfactory, because of discordant values due to shifts. Such an extension, however, indicates a velocity of about 18 ft. per sec. at a gauge height of 16.85 ft. and a corresponding discharge of 41 400 sec-ft.

As a rough check on these values, on June 29th, 1921, a determination of Kutter's n was made for the river channel from the Victoria Avenue Bridge to the Main Street Bridge, a distance of 768.5 ft. At the Victoria Avenue, Union Avenue, and Main Street Bridges, careful levels were taken to the water surface at nine different points on each section. Using the mean of these nine values as the surface elevation at each section, the following results were obtained:

Victoria Avenue, mean water surface	4 662.61
Union Avenue, " " "	4 662.25
Main Street, " " "	4 661.82

Difference in elevation,

Victoria Avenue to Main Street		0.79 ft.
Distance		768.5
Slope	0.79 =	0.001 +
	768.5	

Careful current-meter measurements were made at Main Street and at Union Avenue, and a careful cross-section was taken at Victoria Avenue. The two meter measurements showed a difference of 3.4% (to be expected in cable measurements), and the mean discharge of 2 865 sec-ft. was used. The height on the Main Street gauge was 4.77 ft.

The "hydraulic elements" of the three sections were as given in Table 19.

TABLE 19.

	Victoria Avenue.	Union Avenue.	Main Street.
Area, in square feet	485.7	488.7	542.1
Perimeter, in feet	155.0	155.0	155.0
Discharge, in second-feet	2 865.0	2 865.0	2 865.0
Mean velocity, in feet per second	5.9	5.75	5.38
Hydraulic radius	3.2	3.2	3.5

The mean section computed from the values given in Table 19, has the following elements: Area, 505.5 sq. ft.; discharge, 2 865 sec-ft.; mean velocity, 5.67 ft. per sec.; wet perimeter, 155; $r = 3.26$; and $s = 0.001$.

Solving for Kutter's C and n, these elements give $C = 99.3$, and $n = 0.0182$. Merriman* gives for Kutter's n:

$n = 0.20$ for canals in very firm gravel.
$n = 0.017$ " rubble masonry.

* "Hydraulics," 1906 Edition, p. 282.

This instance is a canal section with a firm gravel bottom and rubble masonry sides, and, as might be expected, n is intermediate between 0.017 and 0.020.

It is found that, for the float measurement of June 3d, 1921, $C = 126$ and $n = 0.016$, using the following elements: Area, 1 505 sq. ft.; mean velocity, 11.8 ft. per sec.; perimeter, 172.0 ft.; $r = 8.75$; and $s = 0.001$.

This is based on the assumption that the slope remains the same for a much higher gauge height and a much greater discharge, which, in turn, assumes a free outflow below. In view of these assumptions and the uncertainty in the measured velocity, it is probable that the value of $n = 0.016$ should only be taken as an indication that n decreases as the flow increases.

If n does decrease at the rate indicated, for a gauge height of 16.85 ft., it should be about 0.014, in which case: Area, 2 300 sq. ft.; discharge, 37 950 sec-ft.; perimeter, 182.5 ft. (approximate); $r = 12.5$; $s = 0.001$; $n = 0.014$; $C = 147$; and $v = 16.5$ ft. per sec.

In applying three different methods, the three values obtained:

(*a*) 45 000 sec-ft. logarithmic discharge curve
(*b*) 41 400 " " extended velocity curve
(*c*) 37 950 " " Kutter's formula

give a mean discharge of 41 450 sec-ft., or, roughly, 40 000 sec-ft., as the maximum which could pass under the Main Street Bridge.

Peak Flow.—The actual peak flow of the flood of June 3d, 1921, is not known, but an estimate may be made in an intelligent way and thus, perhaps, a figure established within certain reasonable limits. It is not known how much of the excess height (above the top of the river channel) was caused by obstructions at bridges and in the area adjacent to the mouth of the Fountain River. High-water marks show that the river rose about 7 ft. above the stone coping which tops the side-walls of the river channel. If the maximum possible carrying capacity of the channel to the top of the coping is taken as 45 000 sec-ft. and to that is added the quantity which could be carried by an additional 7 ft. in depth (which would be, say, 150 ft. wide by 7 ft. deep by 20 ft. per sec. velocity), a total of 45 000 + 21 000, or 66 000 sec-ft., is obtained. This assumes, of course, an unobstructed flow in a channel 7 ft. deeper than the present one, but takes no account of the water which flowed outside the present channel, through the town.

Since, of course, this condition of unobstructed flow did not and could not exist at this gauge height, it may be contended that this velocity of 20 ft. per sec. did not exist and that, as the water backed up, its velocity was decreased. No account, however, has been taken of the quantity of water passing through the town, and it is probable that this effect must have been more than counterbalanced and that the figure of 66 000 sec-ft. is a minimum estimate of the peak.

On June 19th, 1921, a cross-section of the Arkansas River, at a point above the mouth of Dry Creek at the Denver and Rio Grande Mile Post 120 + 4 082, was surveyed by W. A. Balcom, Division Engineer, of the Denver and Rio

Grande Railroad. From high-water marks, the area of this section was found to be 8 610 sq. ft., wetted perimeter, 751.6 ft., and the slope of the river was 0.0028 for 1 200 ft. above the section and 0.0022 for 877 ft. below it. Using a mean slope of 0.0025 with these figures and a value of $n = 0.035$, Kutter's formula gives a discharge for this section of 99 876 sec-ft. Using $n = 0.04$ (which seems to be a more reasonable value), the discharge becomes 86 100 sec-ft., about 14% less.

To this must be added the flow of Dry Creek. Late Thursday night, June 2d, 1921, a local cloudburst flooded Dry Creek, and judging from reports, it carried more water then than at the time of the "big flood". Therefore, estimates made from water-marks may give figures for this first flood. The first flood, a large part of which came from the cloudburst on Dry Creek, however, was recorded on the automatic gauge at Pueblo, and, apparently, caused a rise of about 20 000 sec-ft. in the river. The second flow in Dry Creek, if assumed as equal to the first, and added to the Arkansas River at the Denver and Rio Grande Section, will give 86 100 + 20 000 = 106 100 sec-ft. as the peak flow at Pueblo. (In this connection, it is interesting to note that the figures for Dry Creek indicate a run-off of nearly 300 sec-ft. per sq. mile, the drainage area of the creek being about 70 sq. miles.)

The following drainage areas tributary to the Arkansas River between Canon City and Pueblo, have been obtained from the United States Topographic Sheets:

South Side:			
Pueblo to Boggs Creek	20	sq.	miles
Boggs "	25	"	"
Rock "	67	"	"
Peck "	46	"	"
Rush "	34	"	"
Red "	42	"	"
Ritchie "	40	"	"
Hardscrabble Creek	186	"	"
Coal Creek	28	"	"
Oak "	72	"	"
Chandler Creek	38	"	"
Total	598	"	"
North Side:			
Dry Creek	71	"	"
Turkey "	215	"	"
Beaver "	260	"	"
Eight-Mile Creek	140	"	"
Oil Creek	456	"	"
Total	1 142	"	"
Fountain Creek	930	"	"

Although a drainage area of about 3 000 sq. miles is tributary to the Arkansas above Canon City, it is known that this area did not contribute, appreciably, to the flood, as the automatic gauge on the river at Canon City recorded a maximum flow of about 4 000 sec-ft. It is also known of the areas previously tabulated, that only a few flooded excessively. Fountain Creek is excluded from the discussion for the present, since it joins the Arkansas River below the Pueblo Station.

Water from the greater part of the Turkey Creek drainage was held up by the Teller Reservoir, and that from the Beaver drainage by the Schaeffer Reservoir, until the time of its failure about 10 A. M., on Sunday, June 5th, 1921.

Mr. Wells, Superintendent of the C. F. and I. Company's Arkansas Valley Conduit, who is intimately familiar with the south side of the river and in touch with conditions both through personal observation and reports from his ditch riders, states:

"The flood of June 4th was caused by rain falling east of Rush Creek, which enters the river at Swallows. Very little water came down Rush Creek and there was no water either in Ritchie Gulch or Red Creek. Peck Creek, Rock Creek, and Boggs Creek were the creeks that caused the flood."

This statement is corroborated by the postmasters at Siloam and Wetmore, who state that, although a heavy rain fell in their section, it fell steadily and penetrated into the soil.

The combined drainage areas tributary to the Arkansas River, between Pueblo and Swallows on the south side, aggregate about 150 sq. miles and, including Dry Creek on the north, 220 sq. miles. Even if the entire south side drainage, as far west as Portland, is included, there are only 336 sq. miles (eliminating Turkey Creek and Beaver Creek). If a run-off equal to that of Dry Creek on June 2d, 1921, namely, 300 sec-ft. per sq. mile, is assumed, there would be a combined flow of 100 800 sec-ft.

A rainfall of 12 in. in 24 hours on an area of 1 sq. mile, with complete run-off, is equivalent to 640 acre-ft., or a rate of 320 sec-ft., which is nearly what has been already assumed. Such rainfall is very much higher than any reported during the storm which caused the flood of June 3d, 1921, but it could have occurred over a limited area in the Boggs Flat Section where the storm appears to have been the heaviest. Unfortunately, no accurate records are available from this area.

The authors suggest that protection be provided against a peak flow of 168 000 sec-ft. This is based on the run-off from a storm 40% greater than any recorded, using certain assumptions as to rainfall and run-off; 60% of 168 000 sec-ft. is 100 800 sec-ft. which, again, roughly corroborates the estimate of the peak flow.

The evidence seems to show that the peak flow amounted to approximately 100 000 sec-ft., possibly more, but more likely less, since the assumptions on which this estimate is based, are extremes purposely taken in order to be sure that the figure should be too large rather than too small.

The Fountain Creek flood reached Pueblo sometime after the peak of the Arkansas flood had passed, and its peak was reached at, perhaps, 3.30 A. M.,

Saturday, June 4th, 1921. A preliminary estimate placed this peak at 50 000 sec-ft.

A cross-section opposite the Denver and Rio Grande Mile Post 117, approximately 2 miles above Pueblo, surveyed by its engineers, gives between high-water marks the following: Width = 921 ft.; maximum depth = 9.4 ft.; area = 5 258 sq. ft.; perimeter = 940 ft.; hydraulic radius = 5.6; and slope = 0.0042.

The peak flow of Fountain Creek at this point, calculated by Kutter's formula, with $n = 0.035$, was 46 500 sec-ft., and with $n = 0.040$, 40 100 sec-ft. At that time, there may have been 75 000 sec-ft. in the Arkansas River. This would give a total volume of 120 000 sec-ft. as the combined discharge of the two streams.

Below Pueblo, the largest flood in a tributary stream occurred in the St. Charles River. As this flood was caused by the rain storm which brought about the Arkansas flood, and had a shorter distance to travel, it is likely that its peak reached the main river before that of the main flood. Definite information on this point cannot be secured at this time.

From a cross-section made by Messrs. H. D. Amsley and G. C. Price, the peak flow of the St. Charles is indicated to be about 50 000 sec-ft. If one-half of this flow was coincident with that of the peak in the main river, the combined flow below the St. Charles would be 145 000 sec-ft., to which Chico Creek added its volume.

A cross-section of Chico Creek, taken by Messrs. Amsley and Price, indicates that it carried a little more than 20 000 sec-ft., and the statement of Mr. Edson, Foreman of the North Avondale Milling Company's ranch, fixes the time of the rise in Chico Creek at 4 A. M., June 4th, 1921. This would bring it at about the time of the peak in the Main River at this point, and would give a total of about 165 000 sec-ft.

The Huerfano River did not flood excessively, but, from gauge heights at the Ellis Dam, it is estimated that it contributed from 5 000 to 6 000 sec-ft. Below this point, the tributary streams were not unusually high, although they probably did furnish some excess water. If the sum of the tributaries between the Huerfano and La Junta is estimated as 10 000 sec-ft., the peak at La Junta was 160 000 sec-ft.

At the head of the Fort Lyon Canal, about 3 miles west of La Junta, a cross-section was taken by Messrs. Amsley and Price, and the flow estimated through this section as the peak flood is from 180 000 to 200 000 sec-ft.

At the head of the Amity Canal, near Prowers, certain flood-flow estimates were made by Oscar Hallbeck, Engineer for the Arkansas Valley Sugar Beet and Irrigated Land Company, as shown in Table 20.

The figures given in Table 20 show a peak flow of 170 000 sec-ft. which continued for 3½ hours, with a gradual decline, and a total discharge of, approximately, 200 000 acre-ft. in 24 hours. It is probable that the peak would flatten out on the lower river and that the flow would continue for a longer time, as indicated in the measurements at the Amity Dam.

TABLE 20.

Time.	Date.	Velocity, in feet per second.	Quantity, in second-feet.	Height over dam, in feet.
11.00 P. M.	June 4, 1921.	.....	2 000	.....
12.00 P. M.	"	.....	6 000	.....
1 00 A. M.	June 5, 1921.	.....	10 000	.....
1.10 A. M.	"	.....	15 000	2.5
1.30 A. M.	"	.....	20 000	3.0
1.40 A. M.	"	.....	28 000	4.0
2.00 A. M.	"	20	45 000	5.5
2.30 A. M.	"	20	55 000	6 0
3.00 A. M.	"	25	80 000	7.0
3.30 A. M.	"	25	110 000	7.5
4.00 A. M.	"	30	155 000	8 5
4.30 A. M.	"	30	170 000	9.0
8.00 A. M.	"	.....	170 000	9.0
8.30 A. M.	"	.....	155 000	8.5
9.00 A. M.	"	.....	130 000	8.0
10.00 A. M.	"	.....	130 000	8.0
11.00 A. M.	"	.....	130 000	7 5
12.00 M.	"	.....	110 000	7.0
1.00 P. M.	"	.....	80 000	.....
8.00 P M.	"	.....	.....	.....
9.00 P. M.	"	.....	55 000	6.0

This effect is still further illustrated by measurements not yet available, taken at Syracuse, Kans. At this point, the river did not overflow its channel, but was confined between the abutments of the bridge, in a channel about 800 ft. wide. Mr. Knapp, State Water Commissioner of Kansas, estimates the peak flood at this point as not more than 50 000 sec-ft. (subject to verification). Evidently, the time of passage of the flood must have been considerably longer, if these figures are correct.

Information now being gathered may show minor discrepancies in some of these figures and will doubtless throw more light on the actual area of origin of the flood, but it is believed that, in the main, the conclusions arrived at are correct within reasonable limits and may be of value in considering plans for future flood protection.

Although the greater part of the damage done was in and around Pueblo, it is evident that great losses occurred in the lower valley, particularly to irrigation structures, and a complete flood-protection plan should consider the lower valley as well as Pueblo.

In the writer's opinion, the adoption of a "conservancy district" law, similar to that of Ohio, should be passed as a first step, in order to provide the machinery for raising money for the construction of flood-prevention works. The costs of the necessary works should be assessed in proportion to the benefits to be derived, against cities, towns, corporations (public and private), and all other interests benefited.

No detailed definite study has been made as yet of the possible plans for flood protection, although various schemes have been suggested. A discussion of these plans without adequate information and reliable estimates of cost will not be attempted at this time.

It is due to Mr. Amsley's diligence and foresight in this connection that any reliable figures have been preserved. The measurements made by him June 2d, 3d, and 7th, 1921, and his subsequent work in establishing gauge heights, embrace all the data on record.

Fig. 21 is a hydrograph showing gauge heights and discharge of the Arkansas River, at Pueblo, from June 1st to 7th, 1921. Since the automatic gauge was lost during the evening of the flood of June 3d, 1921, no strictly accurate gauge heights are available from that time until June 6th, but approximations were obtained in various ways, from reliable statements, water-marks, inspection from a distance, counting the number of stones in the side-walls visible above water level, etc.

This graph should be quite accurate except for the time during the peak of the flood when the river was out of its channel. This time is quite definitely fixed, especially the time of the first overflow and the peak flow. The time when the water in the channel was below the stone coping is fixed from the statements of people who spent the night in City Hall. They agreed that the stone copings lining the channel were visible at daylight above the water.

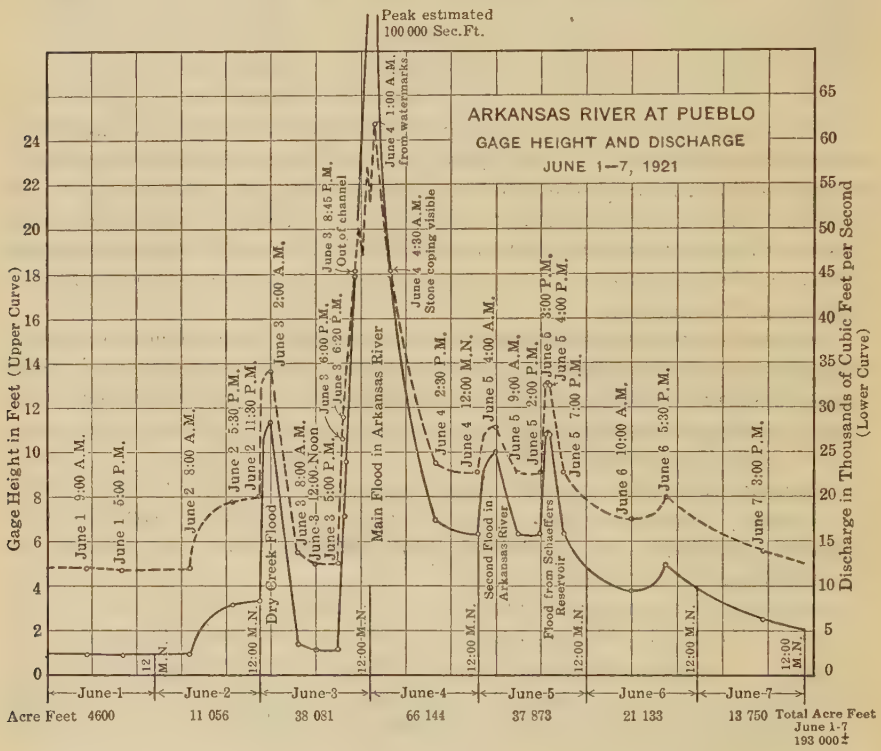


Fig. 21.

An interesting feature of Fig. 21 is the peak representing the flood from the Schaeffer Reservoir which failed about 10.00 A. M., on Sunday. This water reached Pueblo at 2.00 P. M. The total discharge of the river from 2.00 P. M. to midnight, on June 5th, was 15 173 acre-ft. (computed from the diagram). Assuming that the river was falling, from 2.00 P. M. to midnight, at the rate indicated, it would have carried about 11 580 acre-ft. during this time; the difference—3 593 acre-ft.—represents the water from the Schaeffer Dam. The

high-water line capacity of the Schaeffer Reservoir was 3 190 acre-ft., but this was undoubtedly exceeded before the reservoir broke, and this excess between the capacity and the water "accounted for," might be represented by the difference of 400 acre-ft.

The main flood computed from 5.00 P. M., June 3d, to midnight, June 4th, amounted to more than 90 000 acre-ft., or, in round numbers, 100 000 acre-ft. in 19 hours, and is the largest flood in the Arkansas Valley of which there is any record. The total for the week through Pueblo was about 200 000 acre-ft., or more than one-third of the total flow for 1920 at this point.

GEORGE G. ANDERSON,* M. AM. SOC. C. E. (by letter).—The authors have earned the approbation of the members of the Society in presenting the accurate and comprehensive, though concise, statement of the features of the great flood in the Arkansas River in 1921, and the resulting situation in the City of Pueblo and the Arkansas Valley, Colorado. Such approbation is equally due to Mr. Hosea for the able and painstaking manner in which he has presented the facts regarding the flood flows of the river. It was necessary to make clear early that the great volume of the flood, and the extent of the resulting damages, came from entirely natural causes, and in the rehabilitation of the city and of the adjacent territory, serious engineering problems are presented, which call for grave consideration in their solution.

From the testimony of such observers as Mr. Wells (quoted by Mr. Hosea), that the greater part of the run-off, at least on the south side of the river, resulted from the rainfall east of Rush Creek, and in the absence of records of rainfall from a greater number of rain-gauge stations throughout the area covered by the general storm, it may be a warrantable assumption that one-half the volume of 100 000 acre-ft. came from less than 300 sq. miles of drainage area. From information obtained by the writer, while investigating the conditions on Beaver Creek and the upper reaches of Fountain Creek, shortly after the floods, it would appear, however, that intense rainfall occurred on the north side of the river and caused run-off which, undoubtedly, contributed to the flood in some volume.

Mr. Hosea states that the "Beaver Creek drainage was held up by the Schaeffer Reservoir until the time of its failure, about 10 A. M., on Sunday, June 5th, 1921." Of the total Beaver Creek drainage of 260 sq. miles, there are only 136 sq. miles above the Schaeffer Reservoir. On some considerable part of the remainder of 124 sq. miles, there was a very heavy rainfall and a consequent large volume of run-off.

At the Schaeffer Dam a measurement, not by a rain gauge, indicated that 4 in. of rain fell during the night of Friday, June 3d, and the precipitation was stated to have been greater at Beaver Park, about 7 miles southwest. The official record at Florence shows only 0.99 in. of precipitation on June 3d, and only a slight run-off occurred in Eight-Mile Creek during that night. These facts may help to define the approximate western limit of the great storm, although a detail in one element may be pointed out in this connection, and, later, elaborated in a broader aspect of the circumstances as they are now known.

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Although the total precipitation for June 3d, at Florence, as given in Table 5, is 0.99 in., it is unlikely that this amount was the total for that day of the rainfall which caused the run-off accumulating in the flood at Pueblo during the night of June 3d and the early morning of June 4th. It is much more likely that the quantity given as the total for that day at Florence was the rainfall up to 6.00 P. M., which hour is the beginning and ending of the Weather Service Bureau's day at various stations in Colorado, for instance, as at Colorado Springs and Lake Moraine, while, at Victor, the day begins and ends at 4.00 P. M. It is noticeable that on the following day, Saturday, June 4th, at Florence the total rainfall is the maximum quantity, 3.31 in. It is probably correct to state that the greater part of this fell during the evening, after 6.00 P. M., and the night of June 3d, and in all probability thus contributed some volume to the flood at Pueblo and, also, to its peak flow.

Although only a slight run-off was observable in Eight-Mile Creek on the night of June 3d, Brush Hollow Creek, the next creek eastward, discharged a very high run-off. Subsequent measurements indicated more than 6 000 cu. ft. per sec. as the peak flow from a drainage area not in excess of 25 sq. miles. The next creek eastward is Beaver Creek which is known to have increased in discharge below the Schaeffer Dam before that structure failed, but all evidences of the volume on that night have been obliterated.

The next creek eastward is Turkey Creek. Although it may be correct to state that the Turkey Creek Reservoir retained the stream flow which occurred above it, there were evidences of quite heavy rainfall on that area, with considerable damage to roads and irrigation ditches. If that heavy rainfall did not extend into the area which the Turkey Creek Reservoir does not intercept, there is indication that the intensity of the storm varied in different localities, which is quite probable. For instance, on the Dry Creek area, the next eastward to Turkey Creek, there is every evidence of intense rainfall and a large run-off.

From the foregoing comments, the writer is of the opinion that, although the greater volume of the Arkansas River flood came from the south side of that stream, and largely east of Hardscrabble Creek, considerable volumes were added from streams on the north side, and that the tributary area of these streams cannot be wholly disregarded in these considerations.

At the Schaeffer Reservoir the heavy rainfall did not commence until about 7.30 P. M., on June 3d, and the consequent run-off may not have reached Pueblo at the time of peak flow, but, undoubtedly, it did add something to the total volume. The flow of Beaver Creek at and below the Schaeffer Dam did not exceed 90 cu. ft. per sec. until 4.00 A. M., on June 4th, when the water surface of the reservoir reached the spillway level.

The writer is of opinion that the statement, "in the two largest storms [of the Arkansas Valley], namely, those of May, 1894, and June, 1921, the average rainfall increases quite uniformly with the elevation of the drainage area", is apt to be misleading, and to require revision in its application to the storm of June, 1921.

As has been stated, the rainfall at the Schaeffer Dam, on Beaver Creek, at an elevation of 5 700 ft., during the night of Friday, June 3d, was about 4 in.

The rainfall, at Victor (elevation, 9 775 ft.), from June 3d, 4.00 P. M., to June 4th, 4.00 P. M., was 2.08 in. Victor is on the western slope of Pike's Peak, while Lake Moraine, at an elevation of 10 200 ft., and Colorado Springs, at an elevation of 6 500 ft., are on the eastern slope and in the Fountain Creek drainage. At the two latter points, the precipitation is given, as recorded in Tables 4 and 5, from 6.00 P. M. of one day to 6.00 P. M. of the next. Table 21 shows the comparative rainfall, in inches, at these points.

TABLE 21.

Date.	Victor.	Lake Moraine.	Colorado Springs.
June 3, 1921	0.03	0.65	5.00
June 4, "	2.08	3.68	4.40
June 5, "	1.55	1.40	1.26
June 6, "	0.37	0.18	0.42
June 7, "	0.01	0.00	0.01
Total	4.04	5.91	11.09

At these three stations, the total rainfall for five days shows that the lower elevation actually had more than twice as much precipitation as the average of the higher elevations. By analyzing the daily quantities, keeping in mind the different hour to which the report refers, the record shows that prior to 4.00 P. M., on June 3d, 0.03 in. of rain fell at Victor, prior to 6.00 P. M., 0.65 in. fell at Lake Moraine, and prior to 6.00 P. M., 5.00 in. fell at Colorado Springs.

The detailed record at Colorado Springs is much more illuminating as to the character of the storm, and, between rainfall and altitude, to the relation for this particular storm:

	Rainfall, in inches.
June 3d, 1921, 3.30 P. M. to 6 P. M.	5.00
June 3d, " 6 P. M. to June 4th, 2 A. M.	4.20
June 4th, " 2 A. M. to 6 P. M.	0.20
June 4th, " 6 P. M. to June 5th, 6 P. M.	1.26
June 5th, " 6 P. M. to June 6th, 6 P. M.	0.42
June 6th, " 6 P. M. to June 7th, 6 P. M.	0.01
Total	11.09

The total rainfall of 9.2 in. at Colorado Springs from 3.30 P. M., June 3d, to 2.00 A. M., June 4th, is comparable with the rainfall of 2.08 in. reported at Victor for June 4th, which really occurred after 4.00 P. M., on June 3d, and probably continued, as at Colorado Springs, until the early morning of June 4th. A similar comparison applies to Lake Moraine, with the alteration that the daily periods are parallel as previously given.

These three Weather Bureau Stations are fairly comparable, for, apart from being the only stations in the path of that particular storm, they are situated, relatively, in the general line followed by the storm of June 3d,

which apparently was almost directly at right angles to the front line of the mountains, with a northeast tendency. Generally speaking, this is a characteristic of these summer storms on the Colorado Plateau.

This actual reversal of rainfall increasing with altitude is not unusual in storms of this character, locally and colloquially called "cloudbursts", since this storm which caused the Pueblo flood, was on an unusual and exaggerated scale. As shown in Table 5, it is traceable, with varying intensity of rainfall, along the eastern slope of the Rockies, almost entirely throughout Colorado, with some favored localities excepted, as, for instance, Trinidad, and well into New Mexico, as far south as Albuquerque, on that night of June 3d.

Such storms are usual during the summer months, and although they originate within the mountains, ordinarily at not excessive elevations, and cause some precipitation inside the front range, the greater part of the rainfall, as in this case, ordinarily occurs on the high plateaus or plains areas. There are those who declare that "cloudbursts" do not occur above an elevation of 9 000 ft.

Under such conditions, it does not seem to the writer that any sound deductions can be made from "average" rainfalls, say, at elevations of from 6 000 to 12 000 ft. For the purpose of establishing merely the relation between rainfall and elevation, it would seem that the method used in Table 6 and the averages derived therefrom are misleading. Considering the column for June, 1921, it is noticeable that the fourth group, 6 000 to 12 000 ft., includes Colorado Springs, Calhan, and Monument, all of which are stations outside the mountains proper and on the plateau or plains area. This group includes Cuchara Camps, with a rainfall of only 1.10 in. Cuchara Camps is in the mountains, at an elevation of about 9 000 ft., and near the head-waters of the Huerfano, but outside the belt of this particular storm. The Huerfano contributed only a small volume to the flood-waters of the Arkansas River, east of and below Pueblo.

The average rainfall was 6.29 in. at the typical plateau stations in that fourth group, Colorado Springs, Monument, and Calhan, and for the typical mountain stations, Victor and Lake Moraine. These stations are close to one another and were in the general path of the storm. The first group of three lie outside the mountains, and the second group well inside the mountain area. Comparing the plateau stations, there is:

	Elevation.	Rainfall.
Colorado Springs	6 000 ft.	10.66 in.
Monument	7 000 ft.	3.72 "
Calhan	6 500 ft.	4.48 "

In the third group, if Trinidad, plainly outside the belt of the intense storm, with a rainfall of 1.05 in., is eliminated, the average rainfall at Canon City and Florence is 5.08 in., which is slightly more than that of Victor and Lake Moraine, 4 000 ft. higher.

In the second group, at an elevation of from 4 000 to 5 000 ft., Pueblo with 4.21 in. of rainfall in 72 hours, is the only station which should properly be

compared. Ordway, Two Buttes, and Rocky Ford were certainly outside the limit of the storm.

In the first group the stations, Lamar and Los Animas, were also outside the storm belt.

Another set of four groups could thus be set up, as follows:

	Elevation.	Rainfall.
Pueblo	4 000 to 5 000 ft.	4.21 in.
Canon City, Florence	5 000 to 6 000 ft.	5.08 "
Colorado Springs, Calhan, Monument	6 000 to 7 000 ft.	6.29 "
Victor, Lake Moraine	10 000 ft.	4.86 "
Average		6.81 in.

The probable relation of rainfall to elevation in such storms is thus more clearly indicated, and the fact is revealed that, at the lowest elevation, Pueblo, the rainfall was only slightly less than at almost the summit of the drainage area. The much more significant feature is that directly within the path of the storm, the average rainfall from various typical stations from which conclusions might be made, just as fairly, if not more accurately, by way of forecast of the results in any recurrence, is more than twice that which the authors find and on which they base some important conclusions.

The official record of precipitation at Colorado Springs shows the maximum of any station within the storm area, and is the maximum rainfall recorded at any station in the period of 30 years. It is possible and indeed probable that rainfall as intense as the 9.2-in. rain, which fell at Colorado Springs during the 10½ hours, from 3.30 P. M., June 3d, to 2.00 A. M., June 4th, occurred west of Pueblo, in some considerable portion of the Arkansas River area, which district is assumed to have yielded the greater part of its volume to the flood. The absence of a sufficient number of well placed rain-gauge stations within the areas subject to these torrential summer storms has never been so well illustrated as on this occasion. If records had been available from a large number of such stations during this particular storm, much more really satisfactory deductions could have been made in place of the estimates, more or less dependable, than can now be arrived at, on the most conservative basis.

The consequent defects are apparent in the tabulation of assumed percentages of run-off, which, to the writer, appear to be unreliable largely because they again are based on the difference in elevation, rather than on the rainfall that did occur at each elevation and the conditions affecting run-off that prevailed at the time of the storm. It would seem, for instance, that a rainfall of 9.2 in. in 10½ hours at Colorado Springs, more than 5 in. of which occurred in 2½ hours, would have resulted in a greater percentage of run-off than from 3.68 in. at Lake Moraine in the same time. From personal observation in many such "cloudbursts"—not of the extreme intensity of this particular storm—it has frequently appeared to the writer that the run-off was almost complete and immediate. It would also appear that above an

elevation of 7 000 ft., with less rainfall within relatively the same period, the run-off would be less than at the lower elevations. On this occasion, in the region of Victor and Lake Moraine, the precipitation above an elevation of about 10 500 ft. was in the form of snow, and, in one instance at that elevation, the run-off from about 10 sq. miles of drainage area did not exceed 50 acre-ft. per day for 3 days after the storm, while the average precipitation, as previously stated, was 4.86 in.

It may be that because "probably one-half this volume [100 000 acre-ft., which passed Pueblo] came from less than 300 sq. miles of drainage area between Hardscrabble Creek and Pueblo, * * * the storm which caused the flood was far from a maximum". It was a maximum, so far as flood volume of the Arkansas River passing Pueblo during more than 30 years is concerned, and so far as intensity of rainfall in adjacent territory, as at Colorado Springs, is concerned. A greater rainfall was recorded at Canon City during the storm of 1894—5.06 in. as compared with 3.40 in. in 1921. Unfortunately, that is the only station in the Arkansas Valley above Pueblo, at which comparison may be made.

Assuming the accuracy of the judgment that 50 000 acre-ft. came from 300 sq. miles, an equally intense rainfall with an equally great percentage of run-off from 1 000 sq. miles would result in a flood of more than 166 000 acre-ft., not three times the volume of the recent flood. It is conceivable, however, that a rainfall of an intensity equal to that at Colorado Springs (elevation 6 000 ft., 9.2 in. in 10½ hours) could occur over all the drainage area in the Arkansas Valley above Pueblo and below Canon City, all of it below an elevation of 6 000 ft., and from 1 740 sq. miles, in place of 1 000 sq. miles, produce a flood equal to or greater in relative volume than that yielded from 300 sq. miles on June 3d, which would approach a volume "three times that of the recent flood".

Under such conditions, with a total run-off of more than 300 000 acre-ft., it may be reasonable to expect a peak flow of 168 000 sec-ft. in the Arkansas River at Pueblo, and it may be essential to provide for that volume, since it is only 68% in excess of the recent flood, although that is the maximum discharge of record in a period of more than 30 years, following a precipitation which is also the maximum in the same period, with the single exception of the record at Canon City.

The results anticipated from the Fountain Creek drainage area, following on a similar study, are not equally convincing, however. The flood of June 3d in Fountain Creek, at Pueblo, showed a total volume of 50 000 acre-ft. and a peak flow of 50 000 sec-ft., both of which are, apparently, the maximum of which there is any record.

From the whole drainage area of 930 sq. miles, the total discharge of 50 000 acre-ft. is equivalent to an average run-off of practically 1 in. Approximately, one-half of that area is below an elevation of 6 000 ft., and the greatest rainfall occurred at Colorado Springs, practically at that elevation. Therefore, on the basis of the authors' tabulation of assumed percentage of run-off—55%—the volume of the flood would have been due to an average rainfall of 1.8 in. There are four rain-gauge stations within the Fountain Creek area,

Lake Moraine, Monument, Colorado Springs, and Pueblo, and the rainfall at these stations, on the night of June 3d-4th, as nearly as it may be established from the reports, was:

Lake Moraine	3.68 in.
Monument	2.90 "
Colorado Springs	9.20 "
Pueblo	3.09 "
Average	4.72 in.

In order to produce a flood of 164 000 acre-ft., on the basis of a run-off of 55%, the average rainfall would have to be in excess of 6 in. Although such anticipated flood might be possible, it does not seem to be probable, in view of the facts that the recent flood in Fountain Creek at Pueblo was a maximum alike in total volume and peak flow, as was the rainfall at Colorado Springs and other stations, with the single exception of Lake Moraine. Such anticipation, at any rate, cannot very well be based on the related data in the recent experience.

As an incidental item in connection with these estimates or forecasts, it may be noted that, on the same basis, the area of approximately 183 sq. miles between the site of the suggested detention reservoir on Rock Creek and Pueblo, might produce a flood greater in peak flow than the capacity of the channel within the levees, which existed in Pueblo prior to June 3d, 1921.

It may be proper, and permissible, to bear testimony to the accuracy of some of the detailed statements made by the authors and by Mr. Hosea. At the time of original construction in 1910, the capacity of the Schaeffer Reservoir, at the spillway level, was approximately, 3 190 acre-ft. Some silting had occurred in the basin, but reduction from that cause was offset by the storage above the spillway level which had occurred prior to the failure. For 12 hours or more preceding the failure on Sunday morning, June 5th, the discharge of Beaver Creek had ranged from 1 500 to 4 000 sec-ft., or more.

The writer passed through the Lower Arkansas Valley, below La Junta, on the morning of June 4th, finding the contributions to the river flow from tributary streams generally as presented. There was some flood flow in Timpas Creek, immediately west of La Junta, estimated at about 1 000 sec-ft., and that was probably all diverted before its junction with the Arkansas River.

In their consideration of "Reconstruction and Flood Control", the authors, very properly, have not attempted to do more than give a general outline of possible alternative and combined methods of improvement that would prevent similar damage in the future, and only in such general terms will comments thereon be submitted.

It would seem to be inevitable, and it certainly would be desirable, to combine any reconstruction work in the City of Pueblo with necessary plans for some improvement of the conditions along the Arkansas River below Pueblo, where very great damage was sustained by irrigation works. The interests of the city and the adjacent farming district are so interdependent that some plan incorporating improvement of mutual benefit should be devised, if at all

possible. The authors indicate the necessity of some such co-operative plan, indeed, in the remark that

"* * * the enlargement of the channel through Pueblo in order to carry the peak flow of the river would not benefit property interests in the valley below; in fact, it might have the effect of slightly increasing the flood peaks in the lower river."

In view of this statement, and for other reasons readily apparent, the suggestion made by Mr. Hosea that the creation of a conservancy district, as the first step toward the rectification of existing damages and the prevention of a recurrence in the future, is probably the best course to be pursued in consideration of all the interests affected. It is also probable that some combined method of flood detention storage and channel enlargement will prove the only effective means.

The location of a storage reservoir or reservoirs of adequate capacity, would be confined, undoubtedly, to the channel of the streams. It is improbable that any site of sufficient capacity can be found at reasonable cost on tributary streams, while experience in the recent flood indicates without doubt that the great volume of the flood occurs outside the front range, or foot-hills, of the mountains. As pointed out by the authors, the only known available site, on the Arkansas River at Rock Creek Canyon, has distinct limitations, and these limitations are not entirely confined to the feature of the development of adequate capacity. Although the authors' studies show a required capacity of 210 000 acre-ft., it may be concluded that 100 000 acre-ft. is the economic limit, and even that is curtailed by practical considerations which seriously affect its possible advantages.

Two important railroads, the Denver and Rio Grande and the Santa Fe, are located in what would become the basin of the reservoir. Their removal to other locations is inevitable, and may occasion considerable physical difficulty. It is not conceivable that the railroad companies would consent to leave the tracks where they are, although the authors suggest that in the remarks that "only the most unusual floods would submerge the railroads * * * by storing water in the reservoir" and "the effect of submergence in comparatively quiet water might cause less damage than the high velocities under present conditions". It may be definitely concluded that practical railroad managers would not consent to assume such chances.

It may be anticipated, also, that the residents in the lower sections of the City of Pueblo and the irrigation interests in the Lower Arkansas Valley will not readily accept a proposal to construct, by a high dam in the channel of the stream, a reservoir of large capacity in such location as would seem to imply a continuing menace to life and property. Such an attitude would merely be the reaction from recent experiences. When it was overcome another attitude, probably as important, would develop in the probable desire of the irrigation interests to utilize the reservoirs for irrigation purposes for "hold-over storage". The authors have touched briefly on this in their concluding remarks, in which it is stated that "it is evident that the supply for additional irrigation development is very limited". Thorough investigation of the stream flow of the Arkansas River and appropriations for storage supply would show,

it may be surmised, that the available supply is now well nigh exhausted by "live rights" attached to projects of capacity, except under such abnormal conditions as those which prevailed on June 3d-4th. Any effort to develop "hold-over storage" in a flood-detention reservoir would probably further complicate the distribution of the stream flow among the present water consumers. Even though it may be true that the available flow is now exhausted by the claims of such prior appropriators, there is also the possibility that the water impounded in the detention reservoir, under the "hold-over storage" plea, might seriously lessen its capacity for its main purpose of flood detention.

The authors give no encouragement to the proposition that such a detention reservoir be used as storage for part of the domestic supply for the City of Pueblo, for the reason that the basin would soon fill with silt. To that perhaps could be added the inadvisability of depending on Arkansas River water, as a source of water supply, under all conditions, if in no other respect than that of purity.

The greatest reliance for future protection in Pueblo will undoubtedly be placed on channels of adequate capacity throughout the city. It would also be desirable to raise street and other grades as much as possible. The most superficial study of the street profile in relation to the top of the levees along the river and of the contours in the flooded district, is sufficient to indicate that if life and property is to be reasonably free from undue hazard, in addition to ample channel capacity in the future, there is entailed a distinct elevation of street and property grades, with all that these changes will involve.

ROBERT FOLLANSBEE,* M. AM. SOC. C. E., and E. E. JONES,† ESQ. (by letter).—The U. S. Geological Survey has recently completed an investigation of the Arkansas River flood for the purpose of publishing a water supply paper on its hydrologic features. The field work was performed by one of the writers, Mr. Jones, and the office studies and report were prepared by Mr. Follansbee. The following discussion is based on this investigation.

The small areas of intense rainfall and the rapid rise of the river at Pueblo, and almost as rapid fall, were the remarkable features of the Arkansas River flood of June, 1921. Unfortunately, no regular records are available for the areas of heaviest rainfall and an idea of the precipitation can be gathered only from statements of local residents of those areas. A summary of such statements obtained by personal interviews is given in Table 22.

The rain of June 3d began about 1 P. M. in the foot-hills north of the valley. By 3 P. M., it had spread over the upper and middle parts of the valley and between 5 and 7 P. M., had reached the lower end near Pueblo. The hardest rain, on Eight-Mile, Rush, and Rock Creeks, occurred between 3 P. M. and 4 P. M., while near Pueblo it did not occur until 10 or 11 P. M. The rain continued with intermissions until sometime after midnight.

For the 48 hours ending on the afternoon of June 4th, the Weather Bureau records indicate that the general rainfall in the Arkansas drainage basin

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† Denver, Colo.

between Canon City and Pueblo had been from 3 to 5 in., being heaviest in the northern part of the area near Pike's Peak.*

Two principal areas of intense rainfall are indicated from the statements of local residents and the maximum discharges of the tributary streams. The larger area extended from a point a few miles east of Wigwam, near the southern boundary of El Paso County, to the top of the Wet Mountains near Beulah, a distance of 45 miles, and from a point a short distance above the mouth of Rush Creek nearly to Pueblo, a distance of 15 miles. The smaller area, covering the southern slope of the Pike's Peak uplift which forms the northern part of the valley, extended from a point above the Skaguay Reservoir to a point 3 or 4 miles south of the river, a distance of 25 miles, and from Oil Creek to Beaver Creek, a distance of 11 miles. The combined areas cover about 700 sq. miles.

An investigation was made of the tributary streams between Canon City and St. Charles River, a few miles below Pueblo, to determine the maximum discharge of each stream and the approximate time of the discharge. In addition, the maximum discharge of the Arkansas at Pueblo was estimated. The work began on July 6th, 1921, and the high-water marks left by the flood in the various streams could still be readily detected.

The method used was to select on each tributary stream a portion as near the mouth as possible where the maximum discharge had been confined to the channel, and where the channel was sufficiently straight to eliminate as far as possible the effect of the water piling up on the outer side of bends. Cross-sections were selected at the upper, middle, and lower end in each part of stream measured, and stakes were set at the high-water line on each side of the channel. Beginning at the up-stream end, a careful level line was run around the six stakes, closing on the initial stake. Distances on each side between stakes were measured along the high-water line. Between each pair of stakes on opposite sides of the channel the cross-section was carefully measured. Owing to the effect of bends in the channel, the difference in elevation between adjacent cross-sections was not the same, nor was the distance along the high-water line the same. Both distances and differences in elevation between adjacent cross-sections on opposite sides of the channel were averaged to determine the slope between them.

Each cross-section was plotted on a large scale and the area determined by a planimeter. The discharge between the upper and middle sections, and the middle and lower sections was computed by Kutter's formula, using the average area of the two sections. The discharge through all three sections was also computed, using the total slope and the average of the hydraulic factors for all sections. Where the difference in area between the upper and lower cross-sections made a marked difference in the velocity through each section, a correction for velocity of approach was applied to the slope between them. Although the conditions of flow were not the same for all sections, the main

* An investigation of the Colorado Springs records indicates that an error was made in measuring the rainfall at that point, the original figures being 2¼ times too great. As corrected, the rainfall from June 2d to June 6th, was 5.18 in., which agrees closely with 5.91 in. at Lake Moraine and 4.03 in. at Victor, both near-by stations.

TABLE 22.

Location.	Statement regarding rainfall.	Duration of Rainfall: Began.	Ended.
North of Arkansas River.			
Dry Creek near mouth	Cloudburst	4 P. M.	
Dry Creek, Sec. 26, T. 20 N., R. 65 W	Hardest at 10 P. M	7.30 P. M.	
Sec. 27, T. 20 S., R. 65 W., just west of Pueblo	12 in. (measured in concrete box)	Night of June 3 4.	
Teller Reservoir on Turkey Creek	10 in	3 P. M.	Morning, June 4.
Skaguay Reservoir on West Beaver Creek	7.5 in. (measured in bucket, morning, June 5)		
Three miles east of Penrose	7 in. (measured in standard rain gauge); hardest at 9 P. M.	3 P. M.	8 A. M., June 4.
Penrose		7 P. M.	Morning, June 4
Brush Hollow Creek in Sec. 13, T. 19 S., R. 69 W		8 P. M.	Midnight.
Eight-Mile Creek, 5 miles above mouth	10 in. (hardest about 3.15 P. M)	3 P. M.	
Eight-Mile Creek in Sec. 15, T. 19 S., R. 69 W		5 P. M.	11 P. M.
Oil Creek in Sec. 35, T. 18 S., R. 70 W	No cloudburst; ordinary hard rain		
South of Arkansas River.			
Chandler Creek, ½ mile west of Florence	9 in. (measured in bucket); did not extend up Chandler Creek 3 miles	Hardest at 6.30 P. M.	
Hardscrabble Creek, divide southeast of Florence	4 in., water ran over prairie	3 P. M.	
Rush Creek in Sec. 9, T. 21 S., R. 67 W		3 P. M.	11 P. M.
Rush Creek in Sec. 22, T. 20 S., R. 67 W	Cloudburst for 30 min.; hardest rain 2 miles south		
Peck Creek, 8 miles above mouth	Tremendous rain; water ran everywhere	2.30 P. M.	
Peck Creek in Sec. 34, T. 21 S., R. 67 W		4 P. M.	
Cameron and Osteen Arroyos in Sec. 30, T. 20 S., R. 66 W	Five periods of very hard rain	3 P. M.	Midnight.
		4 P. M.	6.30 P. M.
Head of Rock Creek in Sec. 34, T. 21 S., R. 67 W	4 in.; water on level	9 P. M.	Midnight.
Between Rock and Soda Creeks, Sec. 29, T. 21 S., R. 66 W	Horse drowned in open field	3-4 P. M.	
Boggs Flat, about Sec. 35, T. 21 S., R. 66 W	5 in. in 30 min.; 6 in., water ran over prairie	5 P. M.	Morning, June 4.
Blue Ribbon Creek in Sec. 32, T. 20 S., R. 65 W	Hardest at 10 P. M	4 P. M.	
Blue Ribbon Creek in Sec. 2, T. 21 S., R. 65 W	10 in. (hardest at 10 P. M.)	3 P. M.	
Beulah in Sec. 3, T. 23 S., R. 68 W	Hard rain all night	6 P. M.	4 A. M., June 4.

NOTE.—Except where the method of measuring rainfall is stated, the quantities can be considered only as approximate, as the observer had no means of making an accurate measurement.

channels were uniformly free from vegetation, and as the overflow areas were only a small percentage of the cross-section, a uniform value for n of 0.035 was used for the tributary streams.

In Table 23 are shown the results of this investigation for each tributary stream between Canon City and Pueblo.

TABLE 23.—MAXIMUM DISCHARGE OF TRIBUTARY STREAMS BETWEEN CANON CITY AND PUEBLO.

Stream.	Distance above Union Avenue Bridge, Pueblo, in miles.	Tributary from north or south.	Maximum Discharge, in Second-feet:		Drainage area, in square miles.	Time of flood crest.
			Total.	Per square mile.		
Dry Creek	1.8	N.	24 400	283	86	11 P. M., June 2.
Blue Ribbon Creek	4.2	S.	9 130	1 360	6.7	11 P. M., June 3.
Arroyo	4.8	S.	1 910	1 060	1.8	11 P. M., June 3.
Boggs Creek	8.2	S.	15 400	582	26.5	6.30 P. M., June 3.
Arroyo	9.5	N.	9 740	619	15.8	5.45 P. M., June 3.
Rock Creek	9.8	S.	53 900	913	59	9 P. M., June 3.
Peck Creek	11.0	S.	19 400	564	34.4	5 P. M. or 6 P. M., June 3.
Cameron Arroyo	12.0	S.	13 900	1 900	7.3	9 P. M., June 3.
Osteen Arroyo	13.5	S.	9 060	1 160	7.8	9 P. M., June 3.
Turkey Creek*	14.2	N.	9 000	188	48	8.30 P. M., June 3.
Rush Creek	16.8	S.	4 670	238	19.6	5.30 P. M., June 3.
Red Creek	19.8	S.	911	22	40.6	5 P. M., June 3.
Fred Rohr Gulch	20.0	N.	968	104	9.3	5 P. M., June 3.
Ritchie Gulch	22.0	S	920	41	22.6	5 P. M., June 3.
Beaver Creek†	25.0	N.	9 470	45	213	7.30 A. M., June 4.
Hardscrabble Creek	28.8	S.	3 300	19	173	7.30 P. M., June 3.
Brush Hollow Creek	31.0	N.	5 320	243	21.9	8 P. M., June 3.
Eight-Mile Creek	33.5	N.	10 000	154	65	8.30 P. M., June 3.
Coal Creek	34.5	S.	3 720	167	22.3	7 P. M., June 3.
Oak Creek	35.2	S.	2 760	41	68	7 P. M., June 3.
Six-Mile Creek	35.8	N.	1 890	77	24.6	7.30 P. M., June 3.
Chandler Creek	36.2	S.	1 610	118	13.6	7 P. M., June 3.
Oil Creek	40.2	N.	2 510	6	425	11 P. M., June 3.

* Forty-eight square miles is the area of Turkey Creek water-shed below the Teller Reservoir, as little or no water passed the dam during the period of maximum discharge of the creek.

† The discharge given for Beaver Creek is the maximum due to the storm which occurred more than 24 hours before the failure of the Schaeffer Reservoir (9 miles above the mouth of Beaver Creek). At the time of the maximum natural discharge of Beaver Creek, the quantity passing the spillway and waste-gates of Schaeffer Reservoir was computed to be 3 100 sec-ft. To this was added the drainage to Beaver Creek from 68.5 sq. miles between the dam and the mouth. The maximum discharge of Red Creek, the chief tributary, was found to be 4 490 sec-ft., or 93 sec-ft. per sq. mile. Applying this unit run-off to the entire additional drainage area gives a run-off of 6 370 sec-ft., which added to the 3 100 sec-ft. passing the dam, gives a total maximum natural discharge of 9 470 sec-ft. for Beaver Creek at its mouth.

A study of each tributary showed that the first flood crests to reach Pueblo were those from Boggs, Peck, Rush, and Red Creeks, and Fred Rohr and Ritchie Gulches, all within a distance of 15 miles, and that the time of their arrival was between 7 P. M. and 8 P. M. Although the heavy rains continued during the afternoon and evening and a considerable volume of water entered the river, no more flood crests reached Pueblo until late in the evening. From 10 P. M. until midnight, the floods from the tributaries near Pueblo, which were the streams having the greatest flood discharge, reached the city. At the same time the flood crests from the most distant tributaries arrived, which caused the rapid rise in the river at that time. The period of intense rainfall was so short and the slopes of the tributary streams so steep that the period of

crest discharge was short. This is shown by the rapidity with which the river fell after reaching its peak.

The maximum discharge at Pueblo was determined by measuring the high-water cross-section and slope at a point just west of Pueblo and above Dry Creek. The discharge was found to be 83 500 sec-ft. (using $n = 0.030$), and the discharge of Dry Creek at the time of the maximum stage of the Arkansas was estimated as 19 500 sec-ft., or a total of 103 000 sec-ft. as the peak flow of the river at Pueblo. From a hydrograph of the river at that place, based on records compiled by engineers of the Denver and Rio Grande Western Railroad Company and data used by the authors, the total discharge at Pueblo from 8 A. M., June 2d, to midnight of June 5th, was found to be 145 000 acre-ft. Of this quantity, 90 000 acre-ft. was the total flow from noon of June 3d to midnight of June 4th.

The nearness to Pueblo of the areas of intense precipitation, and the fact that the flood crests from the streams nearest Pueblo arrived at the same time as the crests from the more distant tributaries, combined with their unprecedented unit run-offs, made a condition that could only result in a flood of great magnitude.

The maximum discharge of the flood of 1894 was only 40% of the maximum of the recent flood and was caused by rains extending over the entire drainage basin. At the time of the 1894 flood, the *Rocky Mountain News* recorded that heavy precipitation on May 30th and May 31st extended over the Arkansas drainage basin, in the form of snow at the higher elevations—notably Pike's Peak and the mountains in the upper end of the basin near Leadville. On the morning of May 30th, Salida reported that rain had fallen continuously for 36 hours and probably would continue during the night. The storm at that point, for duration and volume, exceeded anything in the memory of the oldest inhabitant. Table 4 shows that at the regular rainfall stations the storm of 1894 was more severe than that of June, 1921, and yet with a storm more widespread and of greater general precipitation, the maximum discharge of the river was less than one-half that of the recent flood. During the storm of 1894, small areas of intense precipitation were apparently absent. The important part played by these small areas so near Pueblo outweighs any general deductions which may be drawn from the relation of elevation to rainfall, in its effect on possible floods in the Arkansas River.

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RAINFALL AND RUN-OFF STUDIES*

By C. E. Grunsky,† M. Am. Soc. C. E.

With Discussion by Messrs. Dana M. Wood, C. F. Marvin, Rudolph Hering, Olin H. Landreth, Thaddeus Merriman, R. A. Hill, Kenneth Allen, L. Standish Hall, H. V. Hinckley, Walter E. Jessup, E. W. Clarke, W. W. Horner, and C. E. Grunsky.

Synopsis

In California it has been necessary, by force of circumstances, to use rainfall records to a very great extent when approximating the water production of water-sheds. There has been made in that State, therefore, perhaps more than anywhere else in the United States, a study of rainfall and of the relation between rainfall and run-off. As early as 1884, the writer, then Chief Assistant State Engineer, began the study of rain distribution throughout California and prepared for the State Engineer Department a rainfall map (published by the State, but now out of print) based on about 200 rain-station records. All these records were expanded to a common 14-year period (1870 to 1884), for which period the greatest number of complete station records were available. A method of combining station records, after first expressing the rainfall of the climatic year in percentage of the normal annual rainfall, and a method of extending short-term records to long periods, are explained in this paper.

Data are presented to show the range of precipitation in climatic years in the central parts of California, and also the frequency of climatic years with various amounts of precipitation. The difference is pointed out between the normal run-off from any water-shed and the probable run-off that is to be expected in a single season in which the rainfall is normal. The effect of altitude on the intensity of rainfall and on the run-off is discussed, and it is shown that the lower temperature at high altitudes diminishes evaporation and, consequently, increases run-off.

* Presented at the meeting of October 5th, 1921.

† Cons. Engr., San Francisco, Calif.

Formulas are also presented for the calculation of maximum storm-water flow from small and from large areas after the maximum rain intensity for various time-periods has been ascertained. A brief reference to evaporation is made, and a table (Table 9) and a formula (Equation (48)) are presented for estimating the evaporation from known mean monthly temperatures.

The deduced formulas are:

Rain intensity:

$$I = \frac{C}{\sqrt{t}} \text{ in. per hour} \quad \ldots\ldots (1)$$

Maximum rainfall in 1 hour:

$$R = 0.129\ C \text{ in.} \quad \ldots\ldots (2)$$

Maximum urban storm-water flow:

$$d_m = 0.645\ a\ I \text{ sec-ft. per acre} \quad \ldots\ldots (17)$$

or,

$$d_m = \frac{5aR}{\sqrt{t}} \text{ sec-ft. per acre} \quad \ldots\ldots (19)$$

Maximum stream flow from large areas:

$$d'_m = 413\ a\ I \text{ sec-ft. per sq. mile} \quad \ldots\ldots (23)$$

or,

$$d'_m = \frac{3\ 200aR}{\sqrt{t}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (24)$$

$$a = \frac{60}{60 + c\sqrt[3]{t}} \quad \ldots\ldots (29)$$

$$d'_m = \frac{25\ 000\ I}{60 + c\ t^{0.33}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (30)$$

or,

$$d'_m = \frac{190\ 000\ R}{60\ t^{0.5} + c\ t^{0.83}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (31)$$

or, fairly approximate:

$$d'_m = \frac{C''\ R}{t^x} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (34)$$

In the foregoing formulas make:

$c = 0.5$ for impervious areas;
$c = 5.0$ for mountainous areas;
$c = 20.0$ for rolling country;
$c = 50.0$ for flat country;
$c = 250.0$ for sandy regions;

$C'' = 3\,500$ and $x = 0.5$ for impervious areas;
$C'' = 3\,300$ and $x = 0.6$ for mountainous areas;
$C'' = 3\,000$ and $x = 0.7$ for rolling country;
$C'' = 2\,100$ and $x = 0.75$ for flat country;
$C'' = 600$ and $x = 0.8$ for sandy regions.

The Wet Season in California.

As the season advances and the fall months pass, the remark is heard on all sides, "I wonder what kind of a winter we are going to have." This implies,

referring now to the Pacific Slope, and more particularly to California, a desire to know whether the winter will be "wet" or "dry"—a wet winter being one in which the rain and snow materially exceed the normal and a dry winter one in which the precipitation is materially below the normal. To those who are not familiar with the climate of the Pacific Coast, it is necessary to explain that this region is not subject to thunder-storms during the spring and summer and that, in consequence, there is very little rain in the six months from May 1st to November 1st. About 90% of the seasonal rain falls in the other six months, with precipitation at its maximum in midwinter.

On the Atlantic Coast, the rainfall is distributed more uniformly throughout the twelve months of the year, the maximum occurring usually in the spring or midsummer months, and there is a corresponding difference in the behavior of the streams which are the recipients of the run-off resulting from rain or melting snow.

It should be recalled in this connection that many of the Western streams —referring to streams which are not fed, in large measure, from underground sources or by the melting of high altitude snowdrifts—go dry, or almost dry, during the summer and fall, although they may have a large flow in the winter and spring. Their seasonal flow, that is, their flow from some time in the fall, say, from October 1st to the end of the following September, is to be ascribed to the rainfall of the climatic or seasonal year. By climatic or seasonal year, the writer refers to the twelve months from some date about the middle or end of one, to the same date of the next, summer season. By common consent the rain-year of the Pacific Slope has come to be considered as beginning with July. This is proper, because the rain which falls in July and August is generally trifling in quantity and is negligible in its effect on run-off, while any rain in September should be counted as affecting the stream flow in the run-off year beginning about the end of that month. For Western conditions, the necessity is thus apparent of discussing the seasonal-annual, or the climatic-year, rainfall in its relation to seasonal annual run-off. A comparison of the calendar-year stream flow with the calendar-year rainfall would be meaningless. Likewise, any comparison of rainfall in one calendar year with that in another calendar year is not only valueless, but absurd.

To illustrate this point, let five consecutive climatic years, Fig. 1, be considered, in the first of which the rainfall has been normal; in the second year, 50% of normal; in the third, 200%; in the fourth, 50%; and in the fifth, again normal. For such seasons, the corresponding stream flow may have been about 100% of normal in the first and fifth seasonal periods of twelve months; about 25% of normal in the second and fourth periods; and 400% in the third period. By calendar years, assuming like quantities of rain each winter before and after January 1st, one-half of the rainfall of the first seasonal year would be combined with one-half of that of the second year, and so on, and, for four calendar years, the rainfall record would appear as 75% of the normal for two years thereof and 125% of normal for the other two. These figures, showing a departure of 25% from normal, instead of 50 to 100%, as in the case of the climatic years, do not give a correct idea of what has happened, and they cannot be brought into any instructive relation to the resulting run-off from the water-shed on which this rain fell. Any deductions attempted

from them pertaining to the relation between annual run-off and rainfall would be misleading.

It is to be hoped that some day this fact will be recognized by the U. S. Weather Bureau, and that the publication of calendar-year precipitation totals will be discontinued with a substitution therefor of climatic-year totals. On the Atlantic Slope, non-compliance with this desirable subdivision of the calendar year has not resulted in the same degree of inconvenience to the engineer as in the West where the need therefor is more obvious. If uniformity of publication is desired, the requirements of the West in this important matter should control. In this respect the Weather Bureau should follow the example of the Water Resources Branch of the U. S. Geological Survey which has long adopted a run-off year beginning with October 1st, a time when throughout the country the streams are ordinarily at their low stages. The climatic year for which meteorological data are desired, should begin at any convenient time between July 1st and October 1st. As run-off due to rainfall

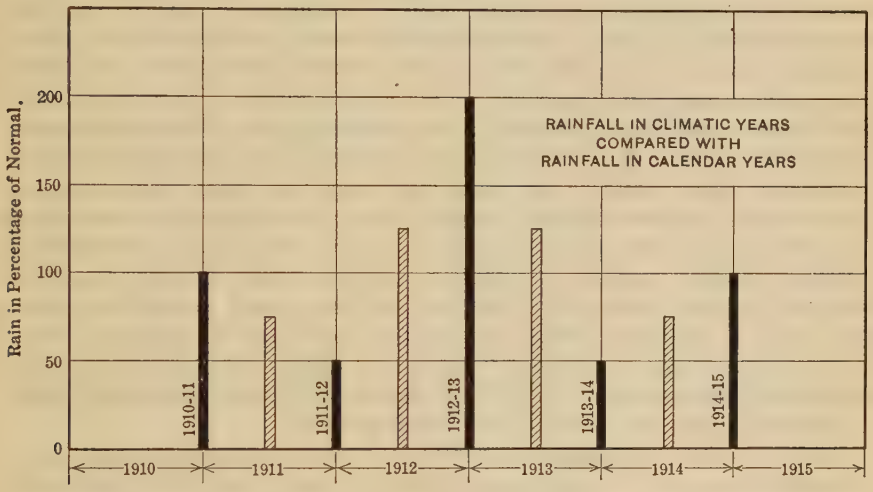


FIG. 1.

is not coincident in time with rain, and as October has been made the first month of the run-off year, it would seem to be desirable to let the "rain" or "precipitation" or "climatic" or "seasonal" year begin with September 1st. It will make no difference on the Pacific Slope whether July and August are placed at the beginning or the end of the climatic year, but as these months are months of heavy rainfall at some points of the East and South, a concession would be readily made, as indicated, to have them placed at the end of the rain-year instead of at its beginning, as is now the practice in California. This, in fact, was the plan, many years ago, of the State Engineer of California, whose compilation of rainfall data was published, in 1886, by the State of California in "Physical Data and Statistics," for climatic or rain-years beginning with September.

If the Weather Bureau declines to make such an innovation, it should at least publish the annual tables applying to Pacific Coast stations in a form

made convenient for use by the addition of a column to its annual tables, in which the totals for the rain-year terminating with June or with August are given.

This paper is intended to point the way to a better understanding of rainfall records and their application to a determination of the water resources and run-off phenomena of any particular region. In order to avoid misunderstandings, however, it will not be amiss to repeat that it has special application to the rainfall conditions of California, which in some essentials are different from those of the East. Thus, for example, it is generally known that wherever in this State the normal annual rainfall (this term being used to include melted snow) is upward of 10 in., an occasional minimum seasonal rainfall of about 30% of the normal (rarely less than 40%) is to be expected and that the maximum may be placed at about 200 per cent. On the Atlantic Coast, no such extreme variation has been observed. The ordinary range there is from about 25% below the annual normal to about 25% above the normal.

California is fortunate in having quite a number of long-time rainfall records, well scattered throughout the State, which are great aids in expanding the knowledge of rainfall to those parts of the State in which the local records cover only short time-periods. Reference should be made in this connection to an earlier paper* on this subject by the writer, dealing with rainfall in the San Francisco Bay region, in which it was explained that one of the characteristics of the cyclonic disturbances which bring rain to the Pacific Slope is the vastness of their extent. The storm the center of which takes a course across Oregon, or even British Columbia, may be accompanied by rainfall as far south as the southern boundary of California. One cyclonic disturbance follows another at intervals of a week or two, but not all of them bring rain. In "dry" winters, that is, in winters of less than normal rainfall, the storm path seems to persist somewhat more to the north than in winters in which the rainfall exceeds the normal. When the causes which fix the general position of the seasonal storm track are discovered, it may be possible to tell in advance whether the winter will be "wet" or "dry."

It is not only a fact that in California the rainfall which produces run-off in material amount is concentrated in the six months from November to the following April, but even during this period, which is frequently referred to as the "wet season", the fair days are more numerous than the rainy ones. Thus, for example, a curve has been constructed to show the frequency of rain for San Francisco. Although this study of rain frequency was made 20 years ago† the result may be accepted as applying to-day. It was found, as shown graphically in Fig. 2, that the average number of days per climatic year (12 months):

With some rain is 66 days;
With more than 0.25 in. of rain is 28 days;
With more than 0.50 in. of rain is 16 days;
With more than 0.75 in. of rain is 10 days; and
With more than 1.00 in. of rain is 6 days.

* "Rain and Run-Off near San Francisco, California," *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 496.

† "Report on a Sewerage System for San Francisco," by C. E. Grunsky, Marsden Manson, and C. L. Tilton.

And that there may be expected:

A day with more than 2.00 in. of rain less than once a year;
A day with more than 3.00 in. of rain once in 5 years;
A day with more than 4.00 in. of rain once in 25 years; and
A day with 5 in. of rain very rarely.

Although such facts as these are of hardly more than local interest, nevertheless, they indicate the scope of studies relating to the weather and to rainfall, which may have a bearing on the water output of a water-shed, or on the maximum storm flow, and for this reason they have been referred to here.

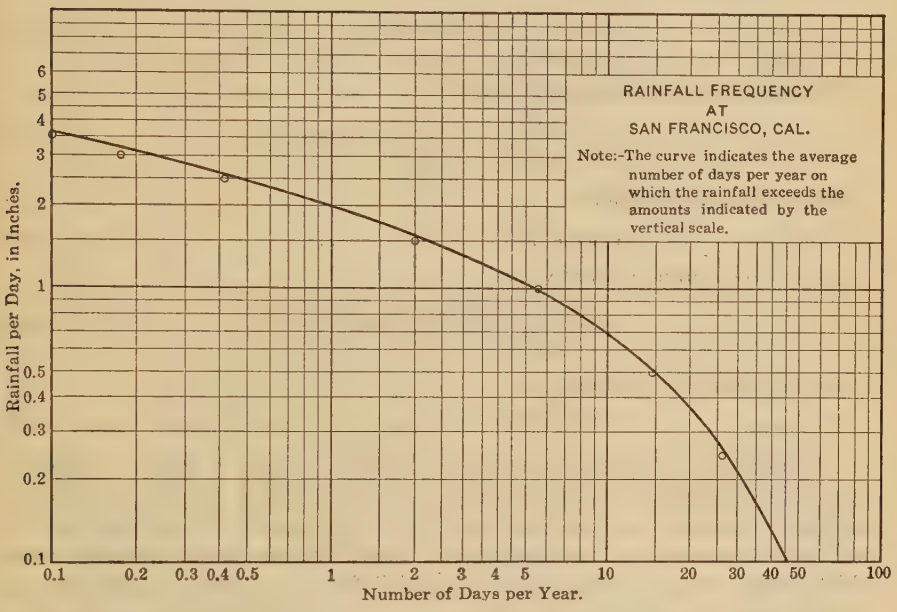


FIG. 2.

INTENSITY OF RAINFALL.

When, in 1892, in connection with the planning of storm-water conduits, information was needed in San Francisco relating to the intensity of rainfall, the Weather Service had nothing to offer except the following: "It never rains in San Francisco an inch an hour, and there is, therefore, no need of a rain intensity record". Needless to say, this idea no longer prevails, and it is now well known that although 1 in. of rain in 1 hour is highly improbable in San Francisco, there are short periods in which that rate of rainfall is greatly exceeded. For any place in the United States, it may be assumed (as amply verified by records of heavy rainfall) that the average rate of rainfall is inversely proportional to the square root of the time to which this rate applies. The following formula, therefore, will be found to give a dependable value of the maximum average rain intensity to be expected in any time-period during which there is heavy rainfall, when observations are

available which disclose the maximum quantity of rain which has fallen in the severest storms in some comparatively short time-period, such as in 30 min., 1 hour, or even in several hours.

Let I = the maximum average rain intensity during t min., in inches per hour.

t = the time, in minutes, to which the rain intensity, I, applies;

R = the maximum quantity of rain to be expected in 1 hour; and

C = a coefficient of definite value for any locality, but not of the same value for different localities.

Then, for maximum intensity:

$$I = \frac{C}{\sqrt{t}} \text{ in. per hour} \quad (1)$$

For maximum rainfall in 1 hour:

$$R = \frac{C}{\sqrt{60}} = \frac{C}{7.75} = 0.129\ C \text{ in.} \quad (2)$$

For intensity coefficient:

$$C = I\sqrt{t} \quad (3)$$

or, for maximum intensity:

$$I = \frac{7.75\ R}{\sqrt{t}} \text{ in. per hour} \quad (4)$$

or, for intensity coefficient:

$$C = 7.75\ R \quad (5)$$

The value of R, that is, the maximum fall of rain in 1 hour, is generally known from observation during heavy downpours. It is, therefore, a simple matter to determine the numerical value of C by Equation (3) or Equation (5) and, with Equation (1), to construct a curve of maximum intensity from which the maximum average rate of rainfall for any length of time, even to 24 hours and longer periods, can be scaled off. To have certain single-station records depart materially from the curve need not be disturbing, because such records are subject to unavoidable error, depending on the direction, force, and character of the wind, uniformity of exposure of the rain gauge to all points of the compass, and other like causes.

It may be noted that, for rainfall conditions similar to those in San Francisco, the value of C in Equation (1) is about 5 and for conditions similar to those in New York City, it is about 15. The maximum rainfall in 1 hour throughout any considerable part of San Francisco will rarely exceed 0.60 in. and, in New York City, it will rarely exceed 2 in.

The diagram, Fig. 3, in which the relation between the maximum average intensity of rainfall for various time-periods, from 1 min. to 5 000 min., and for a number of values of the intensity coefficient, C, is shown, will be found to be helpful first, in determining the value of C when the maximum rain in 1 hour, or some other time-period, is known; and, thereafter, in determining by the aid of this value of C, the maximum intensity of rain in any number

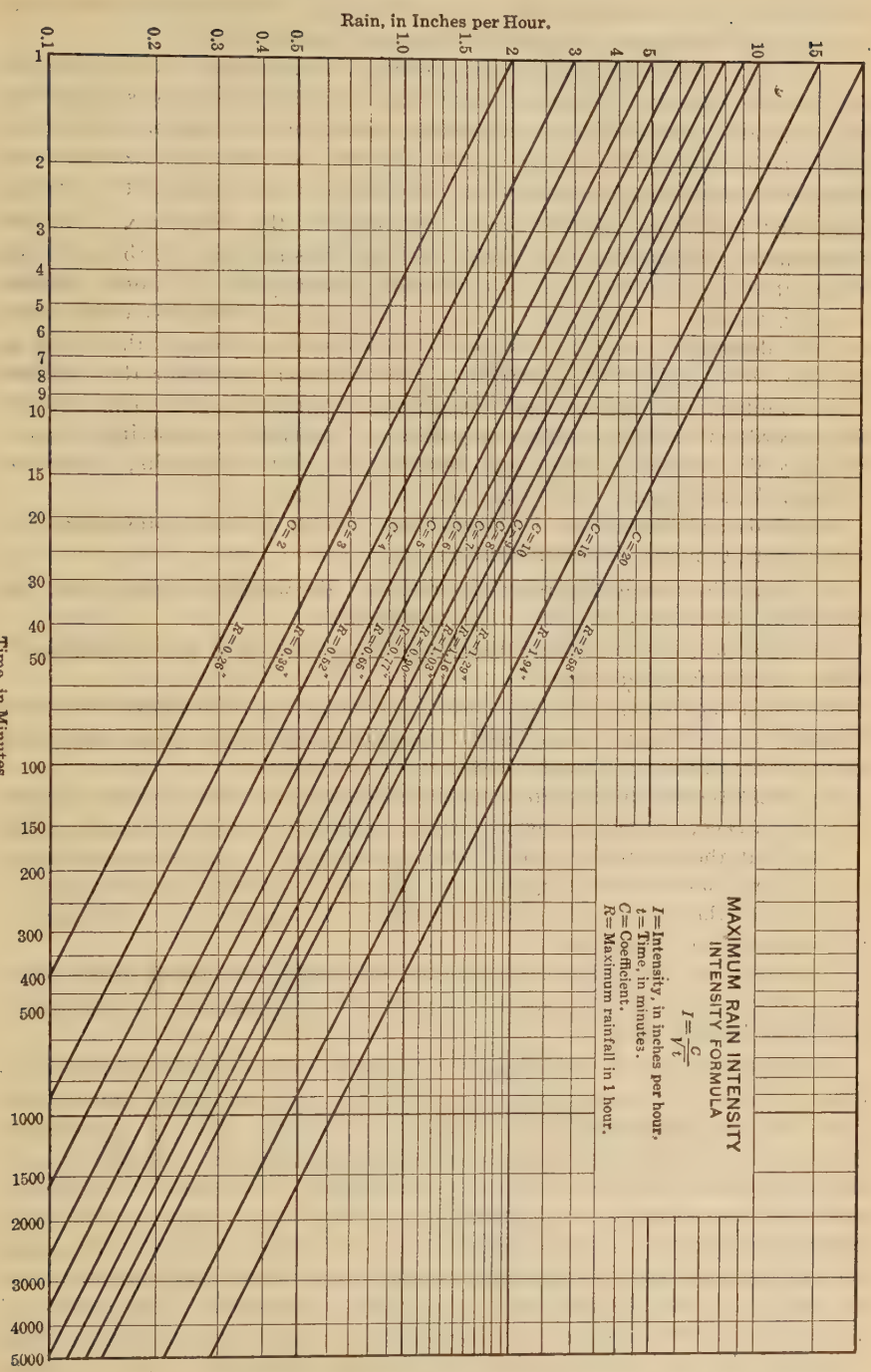

Rain, in Inches per Hour.
0.1
0.2
0.3
0.4
0.5
1.0
1.5
2
3
4
5
10
15
Time, in Minutes.
1
2
3
4
5
6
7
8
9
10
15
20
30
40
50
100
150
200
300
400
500
1000
1500
2000
3000
4000
5000
C=2
C=3
C=4
C=5
C=6
C=7
C=8
C=9
C=10
C=15
C=20
R=0.36
R=0.39
R=0.42
R=0.56
R=0.77
R=0.80
R=1.03
R=1.06
R=1.29
R=1.94
R=2.58
MAXIMUM RAIN INTENSITY
INTENSITY FORMULA
$I = \frac{C}{\sqrt{t}}$
I = Intensity, in inches per hour.
t = Time, in minutes.
C = Coefficient.
R = Maximum rainfall in 1 hour.

FIG. 3.

of minutes. By using logarithmic scales, the lines of intensity of rainfall in Fig. 3, have been made to appear as straight, parallel lines.

It is noteworthy that Fig. 3 lends itself to a modification of the formula for rainfall intensity, Equation (1), which has determined the slope of the intensity lines. If any set of dependable results of observations for various time-periods are platted on the diagram, it may be found to be desirable to give the line limiting the maximum intensities a somewhat different slope from that which has resulted from the use of the factor, $\sqrt{t}$, in the denominator of Equation (1). From any slope thus ascertained, the preferred exponent of t, other than 0.5, can be readily ascertained. It would be well, however, to bear in mind that, owing to the many uncertainties which enter into the determination of the total quantity of rain on an area, this being at variance with single-point observations, there is good reason for the adoption of a formula of the simplest kind. In the same connection, it is to be assumed that records of rainfall applying to points and accepted as applying to areas are more dependable for the longer time-periods, that is, 30 min., 1 hour, or 2 hours, than for the 1-min., 5-min., and 10-min. periods.

In studying the results of observations of rain intensity applying to a water-shed, it should be remembered:

1.—That the rain-gauge record represents only what is happening at the particular point where the rain guage is placed.

2.—That at best the rain-gauge record is only a close approximation of what occurs a few feet away.

3.—That the average of the records of a number of rain gauges will give a better idea of what occurs during the passage of a storm over any place than will be given by a single gauge.

4.—That the maximum rates of rainfall during a storm at various points on a water-shed, no matter how small, do not occur at exactly the same time.

5.—That the maximum rate of rainfall applying to an entire water-shed is necessarily less than that applying to some points within the water-shed.

6.—That the distribution of rain to the various parts of a water-shed is not identical for all storms.

7.—That the mass curves of rain for any storm at various points of a water-shed will show variations in character, that is, sequence of intensities, as well as in quantities.

It follows from such considerations that in the determination of the storm characteristics or rainfall intensities which will give maximum storm-water flow from any water-shed, the single-station record, with its occasionally exaggerated rates of rainfall, at its best, will give only an approximation of the value which applies to the entire water-shed. The plan of combining several station records and of smoothing out the irregular curves that would result from close adherence to such records, is to be recommended. Of course, the fact remains that the maximum possible storm is not likely to be among the particular storms which have come under observation. The records of what has occurred in the past are valuable only as indicating what is probable or possible in the future, and the more complete these records are, the more dependable will be the conclusions which are based thereon.

With due regard to these matters, it will be found that the limiting curve of rainfall maxima in various short time-periods, applicable to any water-shed area, is for all practical purposes, as already indicated, a parabola the elements of which are readily determinable when good local records of rainfall are available.

The possibility will still remain that the occasional storm, exceeding all probabilities and occurring only at long intervals, will produce rainfall of greater intensity than, determined from a limiting curve, the probable maximum. It is well, therefore, when the extreme possible flow is to be estimated, to adopt values for the coefficient in the formula, which provide for some margin over the rainfall intensities that may have been determined during ordinary severe storms.

Various formulas, other than the parabolic type, have been used by engineers to determine the maximum quantity of rain which may fall in any locality in a given time-period. Since the maximum rain intensity is most frequently desired in connection with urban problems, relating to provision for storm waters, the time-period for which it is desired to know the intensity of the rain is generally short. This has led to the quite general use of types of formulas for limiting curves intended to apply to periods of short duration, usually 2 hours or less. Such formulas have no advantage over those of the parabolic type and fall short of requirement whenever large areas are under consideration and it is desired to know the probable maximum rainfall for longer periods of time. Their use should be abandoned either in favor of the parabolic limiting curve, or of some other curve of similar type, in which instead of making t the exponent of one-half in the denominator of Equation (1), some other value approximating one-half is adopted, which will bring the curve into the desired relation to the points indicated by observation.

The best known formula for rain intensity, which departs from the type herein advocated and is intended to be used for short time-periods only, is, as follows:

$$I = \frac{m}{t + n} \qquad (6)$$

in which m and n represent values to be ascertained for each locality to which the formula is to be applied.

The Boroughs of Manhattan and Brooklyn, New York City, have adopted Equation (6), with $m = 150$ and $n = 20$, making

$$I = \frac{150}{t + 20} \text{ in. per hour} \qquad (7)$$

The rainfall in t min., according to Equation (7), will be:

$$R_t = \frac{2.5\,t}{t + 20} \text{ in} \qquad (8)$$

In 1 hour, the probable maximum, as determined by Equation (8), is:

$$R_{60} = \frac{2.5 \times 60}{60 + 20} = 1.875 \text{ in.}$$

For this value on a parabolic limiting curve, it will be found that:

$$I = \frac{14.5}{\sqrt{t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and

$$R_t = 0.242\sqrt{t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The defects of the New York formula will appear from the comparison shown in Table 1.

TABLE 1.—Comparison of the New York Formula for Maximum Rainfall with the Parabolic Formula*

From New York Formula, $I = \frac{150}{t + 20}$; from Parabolic Formula, $I = \frac{14.5}{\sqrt{t}}$.

Time, in minutes.	Rain Intensity, in Inches per Hour, Based on:		Maximum Rainfall, in Inches:	
	New York formula.	Parabolic formula.	New York formula.	Parabolic formula.
1	7.2	14.5	0.12	0.24
5	6.0	6.5	0.50	0.54
10	5.0	4.6	0.83	0.73
20	3.75	3.24	1.22	1.08
30	3.00	2.65	1.50	1.32
60	1.88	1.88	1.88	1.88
90	1.36	1.53	2.04	2.29
120	1.07	1.32	2.14	2.64
180	0.75	1.09	2.25	3.27
240	0.58	0.94	2.32	3.76
300	0.47	0.84	2.35	4.20
360	0.39	0.76	2.37	4.56
720	0.20	0.54	2.43	6.48
1 440	0.10	0.38	2.46	9.12

*It is probable that if the two formulas had been made to agree at some time period in excess of 60 min., making the value of C smaller, the resulting parabolic curve would better fit New York conditions than with C at 14.5.

In the comparison shown in Table 1, the two limiting curves and the two intensity curves have been made to coincide at the 1-hour points. Perhaps if all records of heavy rainfall at New York for time-periods up to 24 hours had been taken into account, a more appropriate parabolic curve for use in that vicinity could have been obtained. The trouble with any formula of the New York type is that it shows nearly as much rainfall for 2 or 3 hours as it does for 24 hours. In this respect, it is obviously defective.

If approximation by the New York type of formula to the maxima for short time-periods should be found to be closer at single-rainfall stations than by a parabolic formula, the question is still open as to whether the latter does not give a more reasonable and better approximation to what is taking place throughout an entire drainage basin the run-off of which is under study.*

Rainfall in the Climatic Year.

Returning now to rainfall in longer periods of time, such as a climatic year of 12 months, it has been found of great convenience in making com-

* "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 294.

parisons to substitute for inches of seasonal rain (that is, precipitation in a rain-year or 12 months), the relation in percentage which the rainfall of the particular 12 months in question bears to the normal rainfall. This is practicable whenever a few long-time rainfall records disclosing a dependable normal afford a good starting point. These records at what may be called primary base stations should cover a time-period of such length that a few additional seasons of either light or heavy rainfall would not materially change the value of the annual normals for these stations.

TABLE 2.—Rainfall at San Francisco and Sacramento, Cal., for the Period 1849 to 1919, Expressed in Percentage of the Normal Seasonal Rainfall

Normal annual rainfall at San Francisco = 22.7 in.
Normal annual rainfall at Sacramento = 19.0 in.

Climatic year, July 1st to the following June 30th.

Climatic year.	Rainfall in Percentage of Normal:			Climatic year.	Rainfall in Percentage of Normal:		
	San Francisco.	Sacramento.	San Francisco and Sacramento, Composite.		San Francisco.	Sacramento.	San Francisco and Sacramento, Composite.
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
1849–50	146	189	168	1885–86	139	170	155
1850–51	33	25	29	1886–87	83	74	79
1851–52	81	95	88	1887–88	74	61	68
1852–53	155	191	176	1888–89	105	105	105
1853–54	105	106	106	1889–90	202	178	190
1854–55	104	98	101	1890–91	78	83	80
1855–56	96	72	84	1891–92	81	80	81
1856–57	88	55	72	1892–93	96	126	111
1857–58	97	79	88	1893–94	81	89	85
1858–59	97	85	91	1894–95	118	127	120
1859–60	99	119	109	1895–96	94	122	108
1860–61	86	82	84	1896–97	102	91	97
1861–62	217	187	202	1897–98	41	55	48
1862–63	60	63	62	1898–99	74	79	77
1863–64	45	41	43	1899–1900	81	106	94
1864–65	108	118	113	1900–01	93	106	100
1865–66	101	94	98	1901–02	84	91	88
1866–67	153	133	143	1902–03	80	87	84
1867–68	171	172	172	1903–04	91	89	90
1868–69	94	85	91	1904–05	103	116	110
1869–70	85	71	78	1905–06	91	126	109
1870–71	62	44	53	1906–07	114	126	120
1871–72	153	121	137	1907–08	76	64	70
1872–73	80	73	77	1908–09	112	115	114
1873–74	105	123	114	1909–10	86	59	72
1874–75	81	95	88	1910–11	112	116	114
1875–76	115	138	127	1911–12	62	50	56
1876–77	44	47	46	1912–13	53	42	48
1877–78	137	123	130	1913–14	130	111	120
1878–79	108	91	100	1914–15	121	91	106
1879–80	116	131	124	1915–16	119	96	108
1880–81	121	129	125	1916–17	70	68	69
1881–82	69	80	75	1917–18	51	56	54
1882–83	87	89	88	1918–19	113	91	102
1883–84	140	124	132	1919–20	46	47	47
1884–85	80	87	84	1920–21	102	89	96

Referring now to California, the records at San Francisco and at Sacramento, beginning in 1849, may be accepted as of this type. These records are presented in Table 2, the rainfall being expressed in percentage of normal. In Column (4) there is given a composite record, a combination of the two separate records, which is a fair index of the rainfall throughout a broad extent of central areas in California.

With the information contained in Table 2, all other rainfall records in the vicinity, beginning with those nearest, and those covering the greatest number of seasons, can be expanded to the full 70-year period of the base stations; and, thereupon, progressing into zones farther removed from the two primary base stations, by the use of secondary base stations, the remaining—often quite fragmentary—rainfall records of the State can likewise be expanded from short-time records of observed rainfall to long-time records of estimated rainfall. Thus, the normal annual rainfall can be determined for all places at which records have been kept. The percentage of normal rainfall noted for any place and season is, at once, an indication of the quantity of rain which fell in the vicinity of that place.

To illustrate the procedure, take Napa where the mean annual fall of rain during thirty-seven seasons, between 1877 and 1919, covered by records, was 24.3 in. The composite record in Table 2 shows that for these same years the mean annual rainfall was 103% of the normal rainfall. Consequently, the normal at Napa is $\frac{24.3}{1.03}$, or 23.6 in., and the Napa rainfall for each year of the 70-year period not covered by actual observation will be found by applying the percentage of normal in the composite column (Column 4) of Table 2 to the normal for this station. The results thus obtained will be practically as dependable as a basis for water production or run-off studies as if there had been actual measurement of rainfall at Napa during the full 70-year period. This is true, because a deduced record of this character, although departing more or less from what did actually take place, nevertheless, fairly represents the probable annual fluctuations in the rainfall and also quite dependably the range from probable minimum to probable maximum. It is possible in this fashion, as stated, to expand the rainfall records for any part of California and thus to produce for any region of the State a dependable long-time rainfall table generally covering the full period from 1849 to 1921.

Frequency of Wet and Dry Seasons.

It is interesting to study probabilities of seasonal rain intensity on the basis of the rainfall records of the past. Taking, for example, the composite record of the two California primary base stations, Table 2, and platting the same as ordinates in the order of their magnitude, a graph such as is shown in Fig. 4, will result.

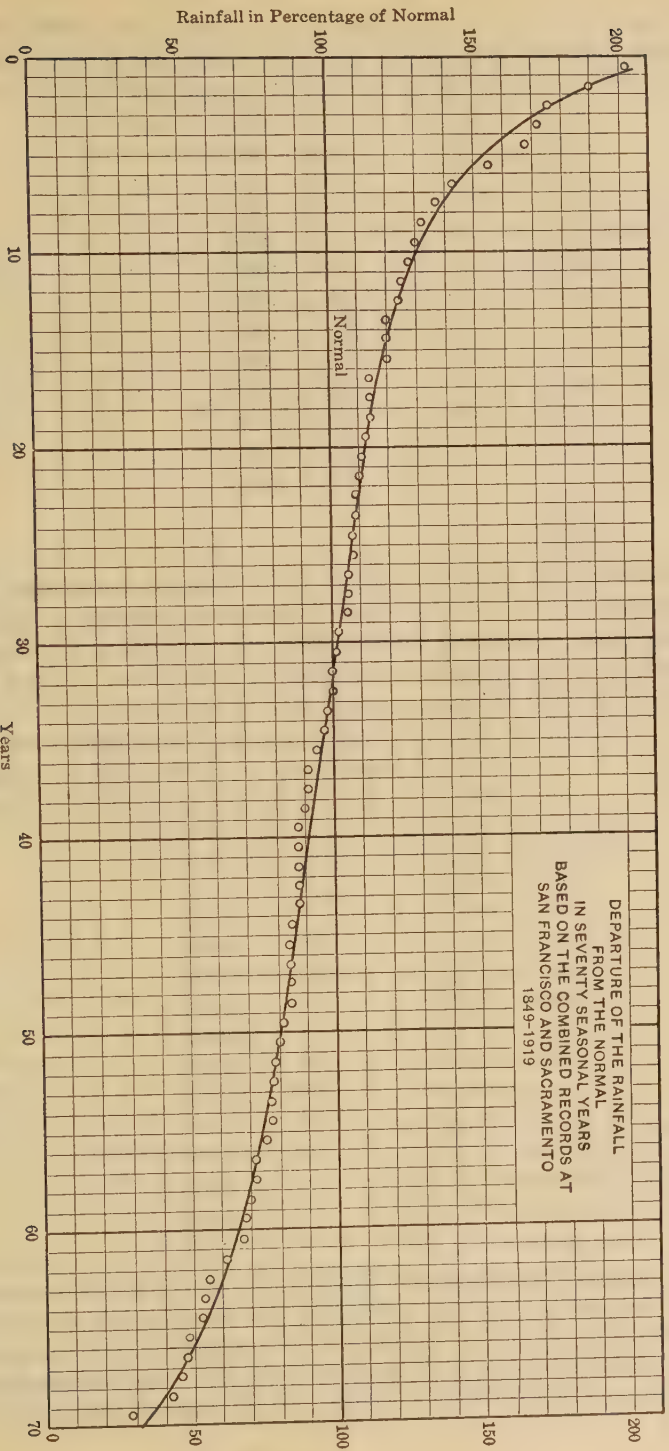

Rainfall in Percentage of Normal
0
50
100
150
200
Normal
Years
0
10
20
30
40
50
60
70
DEPARTURE OF THE RAINFALL
FROM THE NORMAL
IN SEVENTY SEASONAL YEARS
BASED ON THE COMBINED RECORDS AT
SAN FRANCISCO AND SACRAMENTO
1849-1919

FIG. 4.

An analysis of Fig. 4 shows that, in 70 seasonal years, there probably will be no year with less than 30% of normal rain and that there will be:

2	years	with	30	to	40%	of	normal	rain
2 to 3	years	with	40	to	50%	of	normal	rain
3 to 4	"	"	50	to	60%	"	"	"
5	"	"	60	to	70%	"	"	"
7 to 8	"	"	70	to	80%	"	"	"
9	"	"	80	to	90%	"	"	"
9 to 10	"	"	90	to	100%	"	"	"
9	"	"	100	to	110%	"	"	"
7 to 8	"	"	110	to	120%	"	"	"
4 to 5	"	"	120	to	130%	"	"	"
2 to 3	"	"	130	to	140%	"	"	"
2	"	"	140	to	150%	"	"	"
3	"	"	150	to	175%	"	"	"
2	"	"	175	to	200%	"	"	"
70								

or, expanded to 100 years, to express probabilities in percentage, the curve will show that, in the region to which it applies, the rainfall in climatic years will range from a minimum of about 30% to a maximum of about 210% of the normal. In any considerable number of years, when grouped according to the rainfall, there will be:

Less than	30%	of	normal	rain	in	0.5%	of all	the years
30 to	40%	of	normal	rain	in	1.1%	of all	the years
40 to	50%	"	"	"	"	3.3%	thereof	
50 to	60%	"	"	"	"	4.9%	"	
60 to	70%	"	"	"	"	7.3%	"	
70 to	80%	"	"	"	"	10.4%	"	
80 to	90%	"	"	"	"	13.1%	"	
90 to	100%	"	"	"	"	13.6%	"	
100 to	110%	"	"	"	"	12.9%	"	
110 to	120%	"	"	"	"	11.3%	"	
120 to	130%	"	"	"	"	6.6%	"	
130 to	140%	"	"	"	"	3.7%	"	
140 to	150%	"	"	"	"	2.6%	"	
150 to	160%	"	"	"	"	1.9%	"	
160 to	170%	"	"	"	"	1.6%	"	
170 to	180%	"	"	"	"	1.3%	"	
180 to	200%	"	"	"	"	1.9%	"	
More than	200%	"	"	"	"	1.4%	"	

In a broad way it appears from the foregoing and from the diagram, Fig. 4, that in the central parts of California the precipitation in about 26.5% of the climatic years will be within 10% of the normal; that the precipitation in about 55% thereof, will be below normal; and that the precipitation in about 45% will be in excess of normal. It also appears that a climatic year

with less rainfall than 30% of the normal is highly improbable and that there will be occasional seasons with 100% more rain than normal. In diagrammatic form, this information is presented in Fig. 5.

Normal Seasonal Rainfall to Any Date.

As each season or climatic year advances, a comparison with the rainfall to date of a normal season is frequently desirable, and this information, for a limited number of places in California, is being furnished to the public from day to day, through the press, by the local U. S. Weather Bureau forecasters. It has been the practice to compute, on the basis of past records for each station, the normal fall of rain to each date. Instead of this practice, the following graphic method of procedure for regional application is suggested.* For any station or for a composite as already described the mass curve of seasonal rain is platted, using monthly normals and calling the seasonal normal 100%, as shown in Fig. 6. The resulting curve will give for any day

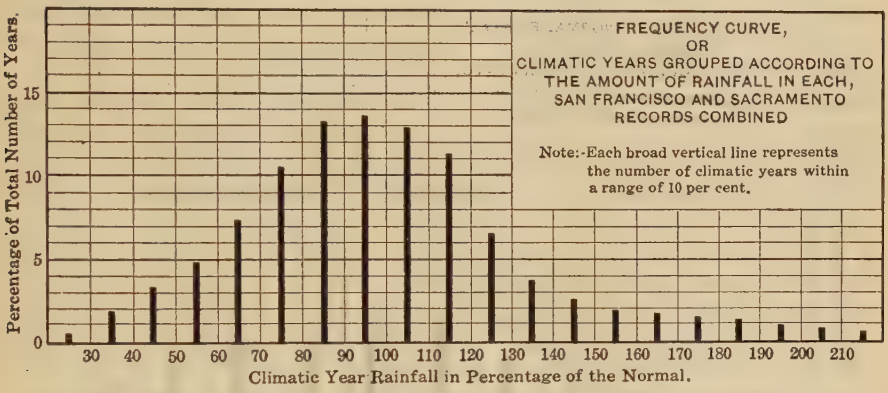

FREQUENCY CURVE,
OR
CLIMATIC YEARS GROUPED ACCORDING TO THE AMOUNT OF RAINFALL IN EACH, SAN FRANCISCO AND SACRAMENTO RECORDS COMBINED
Note:-Each broad vertical line represents the number of climatic years within a range of 10 per cent.
Percentage of Total Number of Years.
0
5
10
15
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210
Climatic Year Rainfall in Percentage of the Normal.

Fig. 5.

of the year the normal seasonal rain to that day in percentage of the annual normal. If, now, the estimated seasonal normal for any station within the region to which a curve of this kind applies, decreases or increases, due to one or more dry or very wet seasons, this change will not materially affect, if at all, the shape of the curve for normal climatic years, but the estimated normal rain to any date will automatically change in the same ratio as the annual normal. A further advantage of using such a mass curve lies in the fact that minor irregularities which appear when the actual mean precipitation to each day of the year is used, are smoothed out, and the true relation of the part-year normal (being the normal to a given date) to the seasonal or full-year normal is more dependably approximated.

The curve, as shown in Fig. 6, being based on the first-class records of rainfall at San Francisco and at Sacramento, is substantially correct for any place in Central California. To illustrate the use of this seasonal mass curve, it may be noted that for February 14th, the normal rainfall applying to that date (being for the season which began on the preceding first day of July) is

* *Journal of Electricity*, Vol. 44, No. 5 (March 1st, 1920).

given by the U. S. Weather Bureau at 14.4 in. for San Francisco and at 12.2 in. for Sacramento. For this date the curve, Fig. 5, shows that 64% of the normal rainfall for a year should have already fallen. For San Francisco, with a normal annual rainfall of 22.7 in., this is 14.5 in. and for Sacramento with a normal annual rainfall of 19.0 in., it is 12.2 in.—values which agree with those determined by the Weather Bureau.

Effect of Altitude on Precipitation.

The attempt is frequently made to establish a relation between the annual rainfall and altitude. A glance at any map showing isohyetose lines will indicate the futility of searching for any such relation that would be widely applicable. It will be found that usually where cyclonic disturbances, accompanied by rain, bring up against, or cross, mountain ranges, the rainfall will be heavier well up on the mountain slope than at the base of the range and that on the

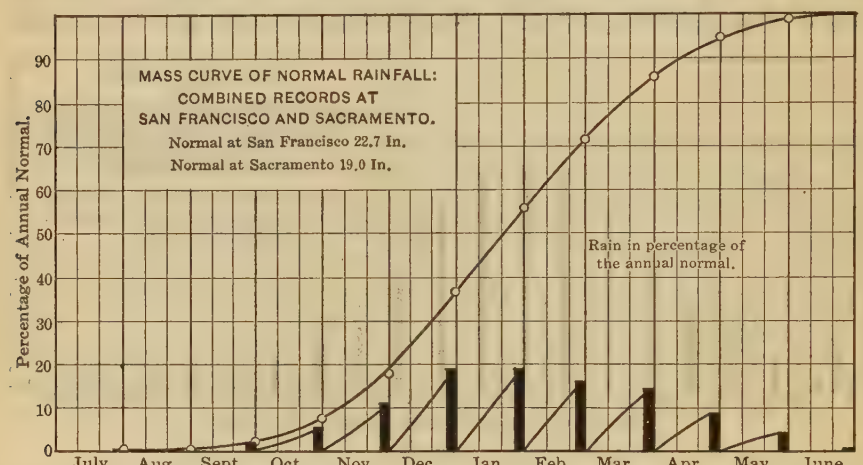


Fig. 6.

far side of the range there will be light rainfall, even though the mountains there may be flanked by a high plateau. The gradient represented by the distance between the isohyetose lines compared with their rain interval, however, may bear almost any kind of a relation to the gradient of the ground surface, and near the summit of the range there is frequently, perhaps it might be said usually, a reversal of the gradient; that is, the region of greatest rainfall is not ordinarily on, but rather somewhat below, the summit of the ridge. It is to be noted, however, that, speaking broadly, the isohyetose lines, or lines of equal quantities of rain, run parallel with the general course of the contour lines of the country. It is, therefore, comparatively easy to extend the isohyetose lines over considerable areas, in regions in which the fall of rain and snow at a few controlling points has been ascertained.

The range in the annual quantities of rain and snow from the western base of the Sierra Nevada, California, across these mountains into Nevada in the latitude of Oroville, increases from about 20 in. in the Sacramento Valley to a greatest precipitation of about 80 in. per annum at elevations about 1 000 ft.

below the crest of the range. On the high plateau to the eastward of the main range, the precipitation drops to less than 10 in. per annum, or only about one-eighth as much as at the same elevation on the western slope of the range.

Run-Off in its Relation to Rainfall.

Closely associated with a study of the rainfall in any region is the study of the run-off which results from the rain. The engineer is interested both in the rate at which this run-off occurs and in the quantity thereof in a considerable time-period, as in a year or in a fraction of a year.

In the study of flood-control projects and in the making of provision for adequate storm-water conduits in urban areas, the engineer is particularly concerned with the maximum rate of run-off which the rainfall on a water-shed can produce. In the study of water-power utilization and the making of stream flow available for irrigation and for domestic and industrial use, the engineer is more particularly interested in the dependable quantity of water produced by a water-shed continuously or during the whole or some part of the year and also with the minimum or low-water flow.

Taking up first, then, the probable annual run-off, or rather the run-off in a period of 12 months, which may reasonably be attributed to the rainfall of the climatic year, it will be found quite practicable to establish a useful relation, expressing probability, between this run-off and the rainfall to which it is to be ascribed. It must not be expected that any law can be pointed out, which will give this relation with accuracy for each individual season. This would be expecting too much in view of the fact that no two seasons even with like quantities of rain in the climatic year are exactly similar in the matter of the sequence, duration, and relative intensity of rain storms. It is readily conceivable that, of two seasons (climatic years), each with the same aggregate or seasonal rainfall, one may be of a type with concentration of rain in a few storms of great intensity, while the other has its rainfall distributed to numerous storms with barely enough rain to saturate the surface soil and with sufficient intervals between storms to permit the soil to lose its water by evaporation. In one case, a relatively large proportion of the rain will find its way to the stream; in the other, very little run-off may occur.

The matter is complicated still further in regions where the rainfall, instead of being concentrated in a certain part of the year, is distributed fairly uniformly to the 12 months. In this event, transpiration as well as evaporation may vary within wide limits, particularly in the warm summer months, being affected by the sequence of rains and the great range in the resulting conditions of sunshine, soil moisture, temperature, wind, and other factors which determine the rate at which water, that would otherwise appear in the stream, is carried off by the atmosphere.

Despite any admission that no dependable estimate of run-off can be made for any single year from the known rainfall conditions which cause this run-off, it is, nevertheless, of the utmost importance to determine the ordinary or probable relation between the rain and its effect on the flow of the stream. It is self-evident that no one can predict the rainfall as to time and quantity for future seasons except on the basis of past records. The same amount and

frequency of departures from the normal or probable conditions are to be expected in the future as have been noted in the past. It is only with full appreciation of this limitation on estimated water yield and estimated maximum and minimum flow, that the relation of run-off to rain should be studied. Otherwise, there will be some disappointment and hesitancy in using results, particularly when station records of rainfall are expanded to long time-periods by the plan advocated by the writer. Nevertheless, for such conditions as prevail in the Pacific Coast States, the method is justified and in most cases the results may be accepted as being quite as dependable as though a complete rainfall record was available for each station for which a fragmentary record has been expanded to a full-period record.

The first rain that falls wets the surface of the ground. A continuation of a gentle rain may be at a rate but little if any in excess of that at which the penetration of moisture into the soil takes place. Thus, considerable rain may fall before there is any material run-off. In the interval thereafter, between one rain storm and the next, the wet ground dries out more or less, its water loss being into the atmosphere by evaporation. If the surface of the ground is kept wet by successive rain storms so that the evaporation, which is greater from a saturated soil than from one that is partly dried out, will be kept at a relatively high rate throughout the time between rain storms, there may be a great loss of water by evaporation. If the rainfall for the climatic year is very light, the time during which the soil is wet and evaporation rapid will be short; consequently, in such years, evaporation will take less of the rain water than it will in wet years. On the other hand, however, if the number of rainy days is great, and the conditions favoring evaporation, by reason of the long time during which the atmosphere is saturated, are poor, then, even with a saturated condition of surface soils, there may be less evaporation than if a fairly regular alternation of a rainy period of a few days with a dry period of similar duration occurred.

It appears from these considerations that evaporation will take more of the water falling as rain in fairly wet years than in dry years and that evaporation (including transpiration) may take from the soil practically all the water which falls in dry years, when the rainfall is very light, leaving nothing to go to the stream. The probable ratio of evaporation to rainfall will decrease as the seasonal quantity of rain increases to some more or less indefinite limit and, conversely, the proportion of rain water which reaches the stream as run-off will increase. This fact was recognized by the writer many years ago and led him to formulate for California the following rule, announced in the earlier paper already referred to, which is applicable in ordinary cases with a fair degree of approximation:

The percentage of the probable run-off due to a rainfall of less than 50 in. in a climatic year of 12 months will be as high as there are inches of rain. The probable run-off due to a rainfall in excess of 50 in. in a climatic year will be 25 in. less than the rainfall; or, expressed by the formula (P = precipitation):

$$r = 0.01\ P^2, \text{ for } P \text{ less than } 50 \text{ in.} \quad (11)$$

$$r = P - 25, \text{ for } P \text{ greater than } 50 \text{ in.} \quad (12)$$

In Equations (11) and (12), P represents the rainfall or precipitation, in inches, in a climatic year, on any water-shed, and r the run-off depth, in inches, over the same area, resulting from this rainfall.

Later studies seem to indicate that quite generally throughout the United States where the precipitation is from 40 to 80 in., the soil and plants growing on the soil will take and dispose of from 22 to 28 in. of water and the remainder will appear in the stream as run-off.

Referring now more particularly to such conditions as prevail in California, the result of the writer's run-off studies early led to the use of two curves, one of which was intended to express the probable relation of run-off to rain in the low or foot-hill regions of the State and the other, showing a somewhat larger proportion of run-off, was intended to show the probable

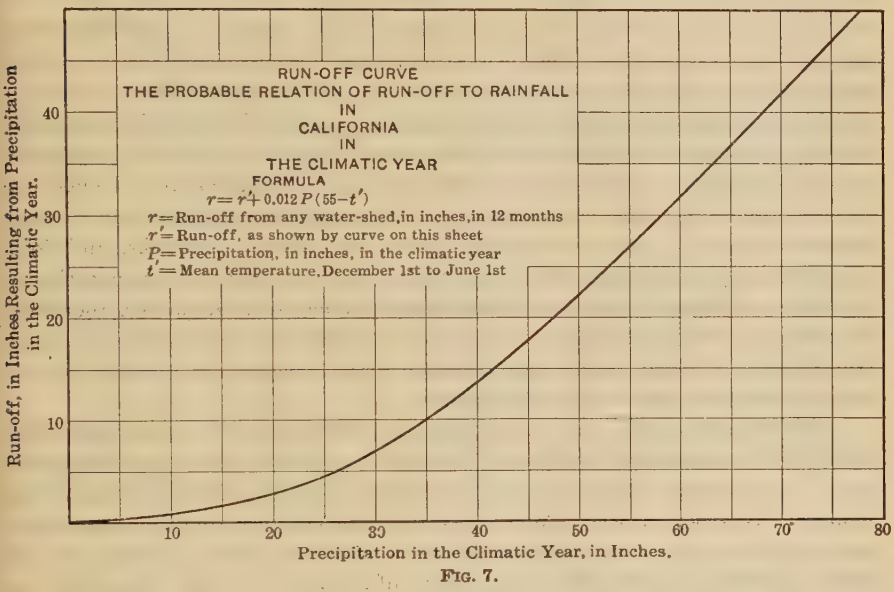


FIG. 7.

relation of run-off to rain in the higher mountain regions of the Sierra Nevada.* It will not be necessary to reproduce these curves which now have only historic value. In revised form, however, a curve is shown in Fig. 7, which takes the place of the original curve for low areas and which shows quite dependably the probable relation between the rainfall of the climatic year and the resultant run-off for California conditions.†

Expressed in tabular form, the writer's run-off curve for water-sheds in California at low altitude gives the values shown in Table 3, which values apply to the run-off resulting from the rainfall in a climatic year of 12 months.

EFFECT OF TEMPERATURE ON RUN-OFF.

It has been found, however, as already suggested that, if applied to high mountain areas, this curve (Fig. 7) would show too little run-off. The cause

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 512.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), p. 1166.

TABLE 3.—The Relation of Run-Off to Rainfall, California Conditions.
(Rain includes snowfall)

Rain, in 12 months of climatic year, in inches.	Resulting run-off, in inches.	Rain, in 12 months of climatic year, in inches.	Resulting run-off, in inches.	Rain, in 12 months of climatic year, in inches.	Resulting run-off, in inches.
5	0.3	24	4.1	55	27.0
10	0.6	26	5.0	60	32.0
12	0.8	28	6.0	65	37.0
14	1.1	30	7.0	70	42.0
16	1.5	35	10.0	75	47.0
18	2.0	40	13.6	80	52.0
20	2.6	45	17.7	90	62.0
22	3.3	50	22.4	100	72.0

for this is not to be sought in the character of the surface of the water-shed, nor yet in the greater altitude of the mountain region, except only as these factors influence temperature and evaporation. All the rain falling on a water-shed, which does not reach the stream, may be considered, as previously stated, to have been returned to the air by evaporation. Some of this rain is returned by transpiration; a small quantity, but generally negligible in its effect on run-off, enters into the composition of tree and plant growth, and a large quantity is lost by evaporation from moist soils, from snowbanks, and from water surfaces of streams and lakes. It is quite apparent that evaporation is the major of these, or the controlling factor; it has seemed proper, therefore, to make an attempt to add a correction increment for temperature or, incidentally, for altitude, to the run-off curve, and a suggestion is made in this regard to be tried out as data accumulate.

It is to be assumed, in any event, that evaporation in so far as it materially modifies the run-off from the Sierra Nevada, or, generally, from high mountain areas, as compared with areas at low altitudes, over a period of about six months, December to May, inclusive (California being especially considered), during which snow is on the ground or the ground surface is moist, will be fairly determinable from the temperature of the air.* It may be noted, too, that any such correction for temperature will represent a larger proportion of the run-off when the precipitation is light than when it is heavy. It is true, however, as already noted, that the aggregate evaporation from areas subject to California winter conditions of wetting by snow or rain must be regarded as increasing with increasing precipitation. These considerations have led to the establishment of the correction factor for altitude, or, better, for the lower temperature due to altitude, as appears in the following formula:

Let P = again the rainfall or precipitation, in inches, during the 12 months of the climatic year;

r' = the depth of run-off, in inches, resulting from the precipitation, P, as shown by the curve, Fig. 7, for areas at low altitude;

r = the depth of run-off, in inches, resulting from the precipitation, P, on any area at any altitude; and

* See the writer's discussion of "Evaporation", *Transactions*, Vol. LXXX (1916), p. 1968.

f = the mean temperature, in degrees Fahrenheit, that prevails throughout the water-shed during the period in which evaporation materially affects the run-off. For California, this period will be from December to May.

The formula may now be written for California, as follows:

$$r = r' + 0.012\ P\ (55 - f) \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

At a temperature of 55° for the months, December to May, there will be no correction to the values shown by the run-off curve, Fig. 7, at sea level; but as the elevation increases and the temperature falls, there will be an increment of water, due to less evaporation, to be added to the run-off.

The foregoing formula, Equation (13), was deduced from the more general expression:

$$r = r' + C\ n\ e\ P\ (55 - f) \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which e represents the increase, in inches, of the monthly evaporation from a water surface, due to an increase of 1° Fahr. in mean monthly temperature (at temperatures of from 40 to 60°); n represents the number of months during which evaporation materially reduces the run-off; and C is a coefficient to be determined by experiment.

For California conditions,* $n = 6$, for the months, December to May, inclusive, $e = 0.1$ in.; and, probably, $C = 0.02$, making $C\ n\ e = 0.012$. To illustrate the use of the curve and formula for the run-off in a 12-month period, the case of a Sierra Nevada mountain area, in California, at an altitude of 6 000 ft., may be taken, on which the climatic year shows a rainfall of 40 in.

From the curve, Fig. 7, the value of r' for 40 in. of rain appears at 13.6 in. In the Sierra Nevada, at altitudes of 6 000 ft., the mean temperature for the six months, December to May, inclusive, is about 40 degrees. Consequently, the seasonal run-off by formula is as follows:

$$r = 13.6 + 0.012 \times 40\ (55 - 40)$$
$$r = 13.6 + 7.2 = 20.8 \text{ in.}$$

It will be noted from this illustration that, due to low temperature at high altitudes, there may be a material increase in the proportion of rain which finds its way to the stream.

Normal Run-Off Computed from Rainfall Records.

After the relation between seasonal (12 months) rainfall and probable run-off has been established by some such curve as that shown in Fig. 7, in figures such as those presented in Table 3, or by the formula, Equation (13), it becomes possible to compute the probable or normal run-off for each 12-month period, for any region, from the rainfall record, when the rainfall is expressed for each season in percentage of the normal rainfall. For all practical purposes and particularly as a means of forecasting future run-off, this method may be accepted with confidence. It leads to a computed record of stream flow which will show what the water output of the stream has been and what water output is to be expected in the future. It shows, too, the relation of the

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1968.

water output in minimum and maximum years to the normal water production and also the frequency of years of ordinary, small, and large water production.

As an illustration, a stream may be selected on the water-shed of which the mean annual rainfall has been found to be 30 in. and the seasonal variation of which in the quantity of rain is represented by the composite rain record noted in Table 2.

It follows that in this water-shed, in a series of 100 years, rainfall and run-off will occur about as set forth in Table 4.

TABLE 4.—Computation of Normal Annual Run-Off for a Normal Seasonal Rainfall of 30 in. from a Water-shed at Low Altitude in a Region for Which the Composite Record of Rainfall at San Francisco and Sacramento is Typical.

Number of climatic years.	Rainfall in percentage of normal.	Annual rainfall, in inches.	Resulting annual run-off, in inches.	Total run-off for period, in inches.
0.5	20	6.0	0.40	0.20
1.7	35	10.5	0.65	1.11
3.3	45	14.5	1.20	3.96
4.9	55	16.5	1.63	7.99
7.3	65	19.5	2.45	17.89
10.4	75	22.5	3.50	36.40
13.1	85	25.5	4.77	62.49
13.6	95	28.5	6.25	85.00
12.9	105	31.5	7.90	101.91
11.3	115	34.5	9.70	109.61
6.6	125	37.5	11.80	77.88
3.7	135	40.5	14.00	51.80
2.6	145	43.5	16.50	42.90
1.9	155	46.5	19.10	36.29
1.6	165	49.5	21.90	35.04
1.3	175	52.5	24.70	32.11
1.9	190	57.0	29.00	58.10
1.4	205	61.5	33.50	46.90
100.	.....	.....	.....	804.58
Mean.....	100	30.0	.....	8.0

According to the calculation presented in Table 4, the normal run-off for a normal seasonal rainfall of 30 in. (that is, within a climatic year of 12 months), is 8.0 in. from a water-shed at low altitude in central parts of California. The probable run-off, on the other hand, resulting from a like quantity of rain, that is, 30 in., during any single climatic year, according to the curve, Fig. 7, or Table 3, is 7.0 in. The normal annual run-off, therefore, from a water-shed at low altitude, throughout which the normal annual rainfall is 30 in., exceeds the probable run-off, due to a like quantity of rain in the single climatic year, by 1 in., or by about 14 per cent. A similar calculation of the normal run-off when the normal rainfall is 20 in. will show an excess over the probable single season run-off, for a like quantity of rain, of about 19 per cent. For a normal seasonal rainfall of 40 in., the excess is 9%; for a normal seasonal rainfall of 50 in., it is 3%; and for a normal seasonal rainfall of 60 in., the normal annual run-off will be about 2% less

than that which is probable in any single climatic year with a rainfall of 60 in.

These relations apply, as noted, to a region in which rainfall conditions are correctly represented by the composite rainfall records, as shown in Fig. 3, applying to certain central areas of California. Where the variation in the rainfall in the several climatic years of a long time-period departs from that shown by this diagram, a different relation between the normal seasonal run-off and the estimated or probable run-off in a season with a rainfall equal to the normal may be found. The numerical illustration is intended merely as a guide in making run-off studies. A different relation will be found, too, when correction factors are applied for altitude or temperature departures from those to which the run-off curve, Fig. 7, applies.

When the run-off is to be determined from a water-shed of so great an extent that there is a wide range in the normal rainfall on different portions thereof, it should be subdivided, preferably in such fashion that the range of normal rainfall in each subdivision will not exceed 5 in., and the estimate of the year's run-off from each subdivision can then be made separately.

The isohyetose lines shown in Fig. 8, for a low to high mountain region lying to the eastward of Oroville, Cal., may serve to illustrate not only the considerable variation of normal annual rainfall within comparatively short distances, but also the difficulty that would exist, in such an area as that covered by the diagram, of estimating the average normal annual rainfall for the entire area from a few isolated station records without recourse to isohyetose lines.

Elements of Uncertainty.

In concluding this discussion of the relation of run-off to rainfall, it may be noted that the problem is complicated by the difficulty always encountered of determining the precipitation on any area of considerable extent, as well as by the lack of precision in the estimates of stream flow. The crudest kind of approximation must usually be resorted to in the matter of rainfall. When water-sheds, for which the shape of the isohyetose lines is known, are studied in relation to rainfall records at a few individual points, the opportunity for error in this particular becomes clearly apparent. In the case of drainage areas such as those in the high mountains of California, for example, there is much uncertainty, too, as to the normal annual snowfall and the snowfall in any individual season. Even for any single point this statement holds good. The distribution of snow over considerable areas with all kinds of exposure to the wind cannot be uniform, and there is uncertainty, too, as to the quantity of water which any given depth of snowfall may represent. The few records of precipitation in California's snowbelt, which are available, are, at best, to be accepted as approximate only, and, yet, it appears that, generally, in respect to precipitation distribution, the information available in this State is as reliable as any elsewhere in the United States.

Maximum Storm-Water Flow from Small Areas.

A complete discussion by the writer of the storm-water flow problem as applied to urban areas will be found in his paper on "The Sewer System

of San Francisco."* The value of the deduced formulas, as there presented, may be reaffirmed, and a wider application may be claimed for them, provided only that fair information is obtainable from past records of rain intensities throughout the affected areas. The element of greatest uncertainty in any formula for the maximum run-off rate will always be the perviousness of the surface of the water-shed. In each case, allowance for the effect of this perviousness must be made by the engineer, on the basis of experience.

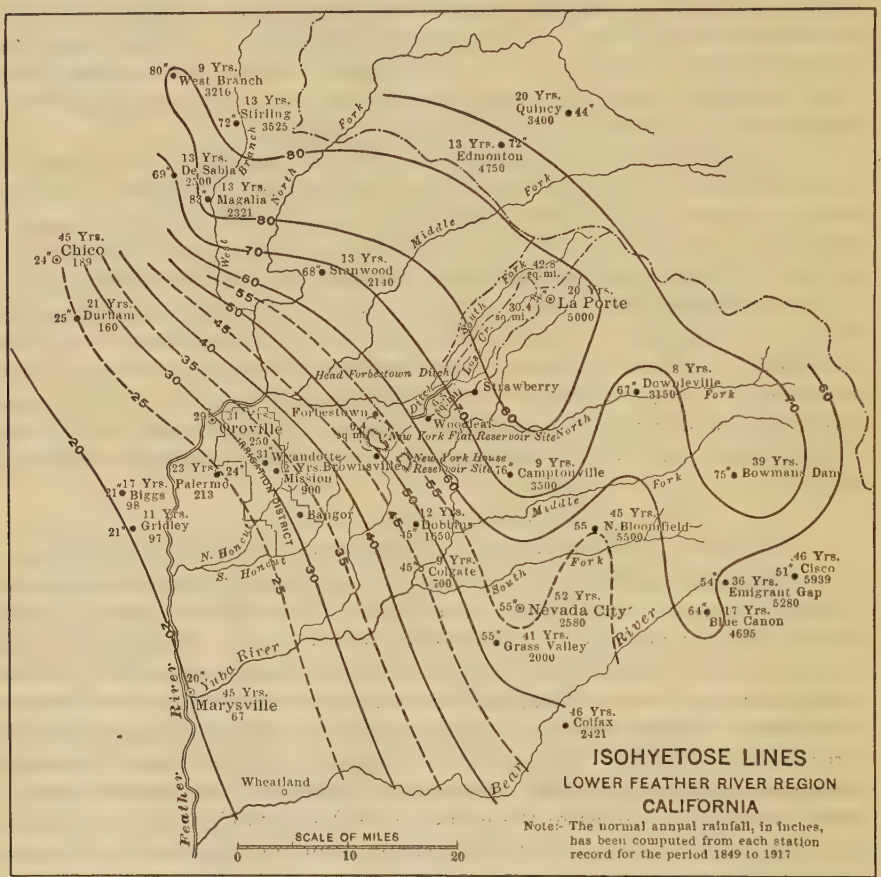


FIG. 8.

Let A_1, A_2, A_3, etc., = the areas of subdivisions of any water-shed, in acres;

A = the area, in acres, of the water-shed;

a, a_1, a_2, a_3, etc., = the coefficients of perviousness, or run-off coefficients, applying to the water-shed and the respective subdivisions thereof;

S = the average slope of the main watercourse, in feet per thousand;

* *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 310.

T = the time, in minutes, that it will take water to flow, under maximum run-off conditions, in natural or in proposed conduits, from the most remote part of the water-shed to the point at which the maximum storm-water discharge is to be determined;

t = the critical time, in minutes, during the continuance of a rain-storm for the area under consideration, being that time within which the rain will produce the maximum rate of run-off.

i = the number of minutes which it takes water to flow on the surface of the area under consideration to the points of entry into the conduits;

d_m = the maximum storm-water flow at the point at which flow is to be determined, in cubic feet per second per acre;

D_m = the maximum storm-water flow, in cubic feet per second.

By the aid of the rain-intensity formulas already noted and of observations throughout the water-shed, which establish the maximum quantity of rain which may be expected to fall thereon in periods of 30 min., 1 hour, 2 hours, or some other period, or periods, of time, the value of the intensity coefficient, C, in the formula for I, Equation (1), namely,

$$I = \frac{C}{\sqrt{t}}$$

is to be determined, and the procedure when the capacity to be given to storm-water conduits is in question, will then be as follows:

Divide the water-shed having an area of A acres into three subdivisions, A_1, A_2, A_3, such that the time required by water to flow from the farther limits of the first over the surface and in conduits of the type which are to be provided, will be one-third, and that the time required for water to flow from the most remote parts of the second subdivision, will be two-thirds of the time required for storm-water to flow from the most remote part of the water-shed to the point at which maximum storm-water flow is to be estimated.

Approximate the time, i, in minutes, that it will take water to flow on the surface of the water-shed to the inlets of the conduit system. Estimate the time, T, in minutes, that it will take water to flow from the most remote part of the water-shed to the point at which maximum flow is to be estimated. Then, determine the critical time, t, from the following:

$$t = 0.40 \frac{T}{a\,A}(a_1 A_1 + 2\,a_2 A_2 + 3\,a_3 A_3) + i \quad \ldots\ldots\ldots\ldots (15)$$

The coefficient of imperviousness, a, for the entire water-shed is found from the following:

$$a = \frac{a_1 A_1 + a_2 A_2 + a_3 A_3}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Consideration must be given to the perviousness of the surface of the watershed in determining the values of a_1, a_2, and a_3. For an entirely impervious surface and relatively small areas, these factors would be unity; for a coarse, well under-drained gravel, they may approximate zero. The values of a, a_1, a_2, and a_3, therefore, will lie somewhere between unity and zero.

For use in a similar formula for maximum flow, the late Emil Kuichling, M. Am. Soc. C. E., based on large experience, recommended* the following:

	Value of a.
For roof surfaces	0.70 to 0.95
" asphalt pavements in good order	0.85 to 0.90
" stone, brick, and wooden block pavements with tightly cemented joints	0.75 to 0.85
" same, with open or uncemented joints	0.50 to 0.70
" inferior block pavements with uncemented joints	0.40 to 0.50
" macadamized roadways	0.25 to 0.50
" gravel roadways and walks	0.15 to 0.30
" unpaved surfaces, railroad yards, and vacant lots	0.10 to 0.30
" parks, gardens, lawns, and meadows, depending on surface slope and character of subsoil	0.05 to 0.25
" wooded areas or forest land, depending on surface slope and character of subsoil	0.01 to 0.20

It is believed, in applying the formulas which are here presented, that it would be better not to attempt the minute classification of surface according to its character, which was suggested by Mr. Kuichling. When small urban areas are under consideration, all surfaces which approximate imperviousness may well be put into one class, because the infiltration of water into the surface material may be neglected when the critical time is short and the rain intensity high.

When larger areas are under consideration, and the soil of the country is an ordinary loam, the approximate values given in Table 5, based on population density, may be used.

TABLE 5.

Population per acre.	Value of a.	Population per acre.	Value of a.
10	0.20	60	0.60
20	0.30	70	0.65
30	0.40	80	0.70
40	0.50	90	0.73
50	0.55	100	0.75

The maximum storm-water flow can now be estimated from the value of I, or from the maximum rainfall, R, in 1 hour, by the formula:

$$d_m = 0.645\ a\ I \text{ sec-ft per acre} \quad \ldots\ldots (17)$$

making,

$$D_m = 0.645\ a\ A\ I \text{ sec-ft} \quad \ldots\ldots (18)$$

* *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 399.

or its equivalent,

$$d_m = \frac{5\ a\ R}{\sqrt{t}} \text{ sec-ft. per acre} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

making,

$$D_m = \frac{5\ a\ A\ R}{\sqrt{t}} \text{ sec-ft} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

In developing this formula, consideration has been given to the effect of the temporary water storage (that is, to the increase or decrease of the volume of water in transit) in the conduits as well as that resting on the surface of the water-shed, on the momentary rate of flow. This is fully explained in the writer's paper* to which reference has already been made.

The value of I is best obtainable from the diagram, Fig. 3, or by Equations (1) to (5). The value of C in the rain-intensity formula, Equation (1), as explained, is dependent on local rainfall conditions. Local records of rainfall, or if no local records are available, then those of some locality with similar meteorological conditions, are to be used in determining the value of C and the resultant rain intensities.

When water-sheds are under consideration, which, though still belonging in the class of small areas, are of so large an extent and so diversified in surface characteristics that the maximum rain intensities in different portions thereof have a wide range, that is, depart widely from the values that would be most applicable for the water-shed treated as a unit, there should be introduced into the calculation the maximum rainfall per hour applying to each subdivision thereof, R_1, R_2, R_3, or the corresponding coefficients, C_1, C_2, C_3, but these should always be determined for the critical time, t, which applies to the entire water-shed.

The formulas may then be written:

$$D_m = \frac{5}{\sqrt{t}} (a_1 A_1 R_1 + a_2 A_2 R_2 + a_3 A_3 R_3) \ldots\ldots\ldots\ldots (21)$$

or,

$$D_m = 0.645\ (a_1 A_1 I_1 + a_2 A_2 I_2 + a_3 A_3 I_3) \ldots\ldots\ldots\ldots\ldots (22)$$

The formulas previously noted for maximum run-off are simple in form and convenient for general use. The numerical factors appearing therein have been deduced in part, as previously stated, from a study of the effect of the constantly changing volume of water actually in transit over the surface and in the conduits, that is, of the effect (on the flow) at the gauging point of an increase or decrease of the quantity of water temporarily in storage within the water-shed. The factors which will always be involved in some uncertainty, are the coefficients of perviousness and of rain intensity.

The following example (based on assumed data) will illustrate the application of the formula for maximum stream flow.

* *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 294.

A topographical survey and preliminary estimates of the velocity at which water will flow in proposed conduits, have established the following values:

A = 2 500 acres
A_1 = 500 acres; a_1 = 0.90
A_2 = 1 200 acres; a_2 = 0.75
A_3 = 800 acres; a_3 = 0.40
T = 85 min.
i = 10 min.

The available rainfall records show, for this water-shed, that during one storm 1 in. of rain fell in 2 hours and, at another time, it rained 2 in. in 6 hours. It is known, too, that the storms in which rainfalls of this intensity occurred, were classed as severe, and that both records may be accepted as approaching single-station maxima. Being for application to a fairly large urban area, the values of R and C deduced from these records will probably be somewhat in excess of the probable value for the whole area, and this excess may be considered as some margin of safety, although not quite enough.

The rain intensity of 0.50 in. per hour deduced from the 2-hour record indicates a value of $C = 5.5$; and the rain intensity of 0.33 in. per hour from the 6-hour record indicates a value of $C = 6.2$.

As the extreme rain intensity may exceed that of either of the only two storms for which dependable records exist, it will be proper to introduce C into the calculation at a value of about 6.5.

Then, by Equation (12):

$$a = \frac{0.90 \times 500 + 0.75 \times 1\,200 + 0.40 \times 800}{2\,500} = 0.66$$

by Equation (11):

$$t = 0.40 \left(\frac{85}{0.66 \times 2\,500}\right) (0.90 \times 500 + 2 \times 0.75 \times 1\,200 + 3 \times 0.40 \times 800) + 10 = 76 \text{ min.}$$

by Equation (1)

$$I = \frac{6.5}{\sqrt{76}} = 0.74$$

and by Equation (15):

$$d_m = 0.645 \times 0.66 \times 0.74 = 0.32 \text{ cu. ft. per sec. per acre}$$

and by Equation (16):

$$D_m = 0.32 \times 2\,500 = 800 \text{ cu. ft. per sec.}$$

This is the estimated maximum storm-water flow from the 2 500 acres.

Maximum Stream Flow or Maximum Rate of Run-Off from Large Areas.

When the capacity is to be prescribed of a spillway for a storage reservoir, or of a stretch of river to which flood-waters are to be confined, larger areas come under consideration than in the case of ordinary urban problems. It is

then desirable to express areas in square miles instead of in acres. With this change of area unit, but again under the assumption that meteorological conditions and, therefore, values of R, the maximum rainfall in 1 hour, and of I, the maximum average rate of rainfall in the critical time-period, t, may be considered to be uniform throughout the water-shed, there will be:

From Equation (17):

$$d'_m = 413\ a\ I \text{ sec-ft. per sq. mile} \quad (23)$$

from Equation (19):

$$d'_m = \frac{3\ 200\ a\ R}{\sqrt{t}} \text{ sec-ft. per sq. mile} \quad (24)$$

from which,

$$D_m = 413\ a\ M\ I \text{ sec-ft.} \quad (25)$$

or,

$$D_m = \frac{3\ 200\ a\ M\ R}{\sqrt{t}} \text{ sec-ft.} \quad (26)$$

in which M represents the area of the water-shed, in square miles.

In applying this formula to large areas, it must be remembered that the individual station records of rainfall do not represent intensities as dependably for large areas as they do for small areas. Whenever practicable the relation of the rainfall on the entire water-shed to that of the single station or, better, to the combined records at a number of stations, should be ascertained. This is done by comparing the station records with the rainfall on the whole area, as determined by isohyetose lines.

The factor, R, in the writer's formulas for the maximum rate of run-off, should always represent an average value for the area to which it applies. That is, if the maximum rain to be expected in 1 hour has been determined for numerous places regularly distributed throughout the area, the value of R to be used in the formula will be the mean of all such determinations.

When there is a wide range in the meteorological conditions in the subdivisions of a water-shed (which need not be restricted to only three), and there is a wide range in the values of R and of I in different parts thereof, it may be advisable to use the formulas in a more general form, deduced from Equations (21) and (22). They can then be written:

$$D_m = \frac{3\ 200}{\sqrt{t}} (a_1\ M_1\ R_1 + a_2\ M_2\ R_2 + a_3\ M_3\ R_3) \text{ sec-ft.} \quad (27)$$

or,

$$D_m = 413\ (a_1\ M_1\ I_1 + a_2\ M_2\ I_2 + a_3\ M_3\ I_3) \text{ sec-ft.} \quad (28)$$

in which, as explained, $M_1 + M_2 + M_3 = M$, and all these areas are expressed in square miles.

Due to the fact that the absorption of water by soil or other pervious material, as well as the rate of evaporation, may be regarded as fairly constant, while the average intensity of the rainfall decreases as the time to which the intensity applies increases, the relative effect of perviousness will increase in some measure as the critical time increases. This statement has, of course,

particular application to large water-sheds in which the critical time may be measured by days with occasional cessation of rainfall instead of by minutes and hours, as is the case in urban problems. The coefficient of perviousness, that is, a in the formula for urban areas would better be considered as a run-off coefficient and should decrease as the critical time increases. Bearing in mind that $a = 1$ for small impervious areas, the following expression for a has been found to give results in fair conformity with observed facts:

$$a = \frac{60}{60 + c\sqrt[3]{t}} \qquad (29)$$

In this case, c may be regarded as a supplemental run-off coefficient with a wide range in value, being almost negligible for impervious areas and increasing with increasing perviousness. A value should be assigned to c with due regard to surface conditions of the water-shed. The following values of c are tentatively advanced for large drainage areas:

For impervious areas	$c = 0.5$
For mountainous areas	$c = 5.$
For low rolling country	$c = 20.0$
For flat country (ordinary soil)	$c = 50.$
For sandy regions	$c = 250.$

These values of c, intended to apply to conditions as they ordinarily obtain in temperate climates, may be found to be too large, giving maximum run-off rates which are too small in localities where the ground may be frozen, or water-logged, or where the maximum run-off rate occurs when rain falls on snow.

In Table 6, some values of the run-off coefficient, a, for various values of c, are given.

TABLE 6.—Values of the Run-Off Coefficient.

Based on: $a = \frac{60}{60 + c\sqrt[3]{t}}$

Time, in minutes.	Impervious areas, $c = 0.5$.	Mountain areas,* $c = 5$.	Rolling country,* $c = 20$.	Flat country, $c = 50$.	Sandy regions, $c = 250$
1	0.99	0.92	0.75	0.55	0.19
8	0.98	0.86	0.60	0.37	0.107
27	0.98	0.80	0.50	0.29	0.074
64	0.97	0.75	0.43	0.23	0.057
125	0.96	0.71	0.37	0.19	0.046
216	0.95	0.66	0.33	0.17	0.039
343	0.94	0.63	0.30	0.15	0.033
512	0.94	0.60	0.27	0.13	0.029
729	0.93	0.57	0.25	0.118	0.026
1 000	0.92	0.55	0.23	0.107	0.023
1 321	0.91	0.52	0.21	0.098	0.021
1 728	0.91	0.50	0.20	0.091	0.019
2 197	0.90	0.48	0.19	0.084	0.018
2 744	0.90	0.46	0.18	0.079	0.017
3 375	0.89	0.44	0.17	0.074	0.016
8 000	0.86	0.37	0.13	0.057	0.012
15 625	0.83	0.32	0.11	0.046	0.010
27 000	0.80	0.29	0.091	0.038	0.008

*Mountain areas are assumed to have considerable rock surface, or rock thinly covered with soil, and rolling country some parts that are considerably less pervious than ordinary soil.

By inserting in Equations (23) to (26), the value of a, Equation (29), and using round numbers, the formulas for maximum run-off are as follows:

$$d'_m = \frac{25\ 000\ I}{60 + c\ t^{0.33}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (30)$$

or,

$$d'_m = \frac{190\ 000\ R}{60\ t^{0.5} + c\ t^{0.83}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (31)$$

and,

$$D_m = \frac{25\ 000\ M\ I}{60 + c\ t^{0.33}} \text{ sec-ft.} \quad \ldots\ldots (32)$$

or,

$$D_m = \frac{190\ 000\ M\ R}{60\ t^{0.5} + c\ t^{0.83}} \text{ sec-ft.} \quad \ldots\ldots (33)$$

The use of the formula is made convenient by Table 7 and by a diagram, Plate IV. In both Table 7 and Plate IV, the value of the factor is shown which, when multiplied by the maximum rainfall in 1 hour, R, expressed in inches, will give the maximum run-off rate, in second-feet per square mile.

The value of t is determinable, as has been explained, from the topographic features of the water-shed and the character of the storm-water conduits. The value of c will be adopted from known characteristics of the ground surface, with special regard to perviousness.

The formula is based on the assumption that $I = \frac{C}{\sqrt{t}}$ (Equation (1)). The appearance of I in the formula is not necessary when R appears therein.

It will be seen by inspection of Plate IV, that straight lines substituted for the curves there shown would afford a fairly good approximation to the values given by the formula. A coefficient, which may be called C'', and the exponent of t, will then be subject to selection according to the surface characteristics of the water-shed. The approximation formula for maximum run-off, in second-feet per square mile, may then be written:

$$d'_m = \frac{C''\ R}{t^x} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (34)$$

where,

$C'' = 3\,500$ and $x = 0.5$ for impervious areas;
$C'' = 3\,300$ and $x = 0.6$ for mountainous areas;
$C'' = 3\,000$ and $x = 0.7$ for rolling country;
$C'' = 2\,100$ and $x = 0.75$ for flat country; and
$C'' = 600$ and $x = 0.8$ for sandy regions.

Whenever large areas are under consideration, that is, when maximum stream or river flow, as distinguished from the storm-water flow of urban conduits, is to be determined, the value of R should be estimated from the maximum rainfall in periods of 24 hours or more, rather than from the maximum rainfall in shorter periods.

TABLE 7.—THE VALUE OF THE FACTOR, $C' = \frac{190\ 000}{60\ t^{0.5} + c\ t^{0.83}}$, FOR VARIOUS VALUES OF t AND c, IN THE FORMULA, $d'_m = \frac{190\ 000\ R}{60\ t^{0.5} + c\ t^{0.83}} = C'\ R$.

Critical time, t, in minutes.	$c = 0.5$, impervious area, in second-feet.	$c = 5$, mountains, in second-feet.	$c = 20$, rolling country, in second-feet.	$c = 50$, flat country, in second-feet.	$c = 250$, sandy regions, in second-feet.
1	3 140	2 920.0	2 380.00	1 730.00	610.00
8	1 100	960.0	671.00	425.00	120.00
27	595	488.0	304.00	174.00	45.00
64	383	297.0	170.00	91.00	20.50
125	272	199.0	106.00	55.00	12.10
216	205	144.0	72.00	36.00	7.80
343	161	108.0	51.00	25.00	5.70
512	131	84.0	38.00	18.20	4.10
729	109	67.0	29.00	13.80	2.90
1 000	92	55.0	23.00	10.70	2.30
1 321	79	45.0	18.70	8.60	1.86
1 728	69	38.0	15.20	6.90	1.49
2 197	61	32.0	12.70	5.70	1.22
2 744	54	28.0	10.70	4.80	1.02
3 375	48	24.0	9.10	4.00	0.86
8 000	30	13.3	4.60	2.00	0.42
15 625	21	8.2	2.70	1.16	0.24
27 000	15	5.5	1.75	0.74	0.15

NOTE.—To find the maximum run-off per square mile in second-feet multiply the figures in Table 7, by the value of R, that is, the maximum rainfall in 1 hour expressed in inches.

To show the effect of area on the maximum stream flow or maximum rate of run-off, areas of similar outline and similar topographical features and subject to the same meteorological conditions may be compared with each other.

Because areas of various extent, but similar in topography and outline, will have values of t fairly proportional to the square root of the surface area, it may be assumed for such areas that,

$$t = K \sqrt{M} \qquad (35)$$

in which K is a coefficient that can be determined for any set of water-sheds complying with the condition of similarity.

Let it now be assumed that for a number of mountain water-sheds of similar characteristics, a value of $K = 20$ has been ascertained. This will make for this special case:

$$t'' = 20 \sqrt{M}$$

and from Equation (23),

$$d''_m = \frac{715\ a\ R}{(M)^{\frac{1}{4}}}$$

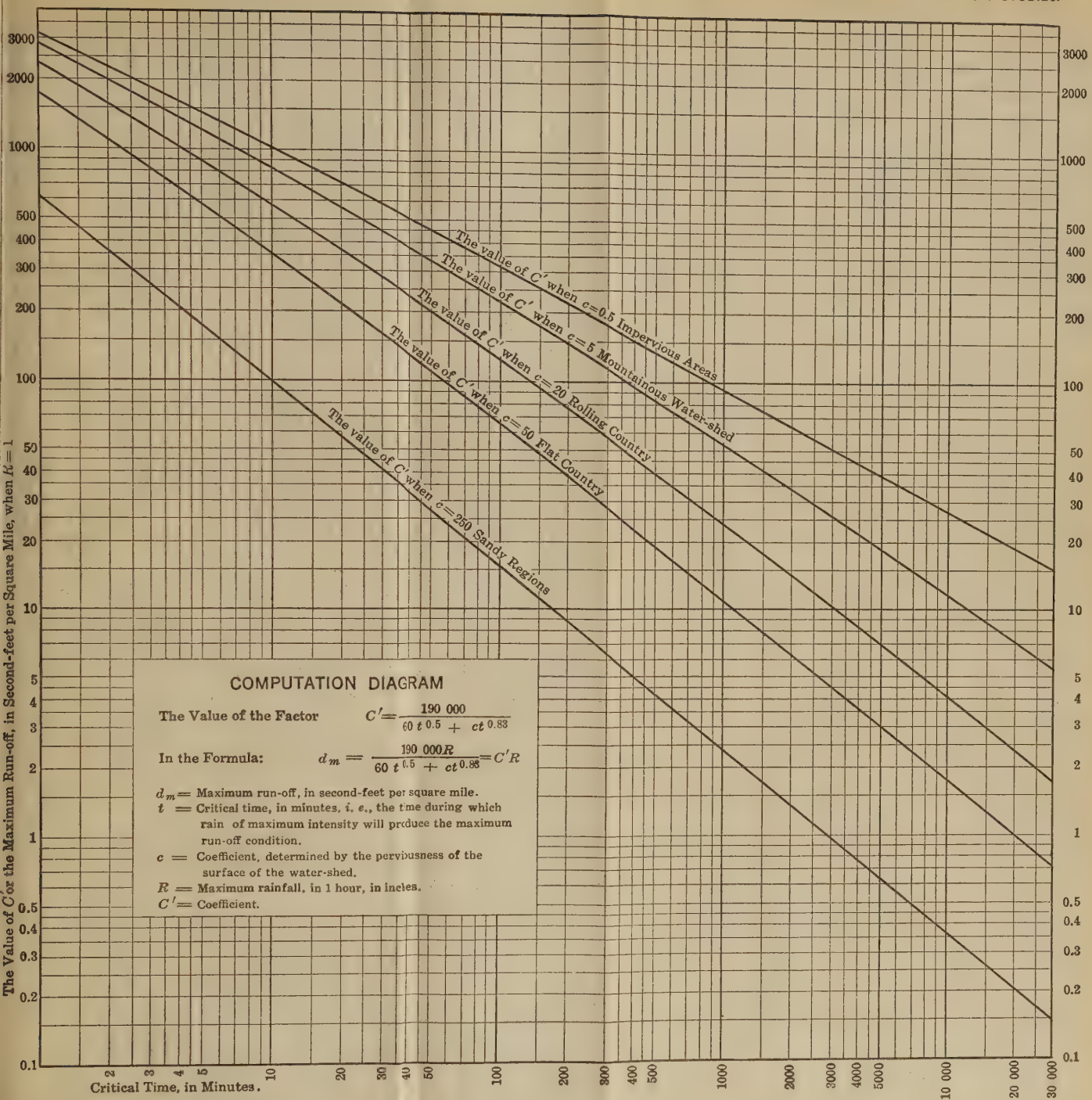
Mountainous regions being under consideration, there will be:

$$a = \frac{60}{60 + 5 \sqrt[3]{t}}$$

Therefore, inserting the value of t'',

$$a = \frac{60}{60 + 14\ M^{0.17}}$$

for the special case only.


The value of C′ when c=0.5 Impervious Areas
The value of C′ when c=5 Mountainous Water-shed
The value of C′ when c=20 Rolling Country
The value of C′ when c=50 Flat Country
The value of C′ when c=250 Sandy Regions
COMPUTATION DIAGRAM
The Value of the Factor C′=190 000 / (60 t^0.5 + ct^0.83)
In the Formula: d_m = 190 000R / (60 t^0.5 + ct^0.83) = C′R
d_m = Maximum run-off, in second-feet per square mile.
t = Critical time, in minutes, i. e., the time during which rain of maximum intensity will produce the maximum run-off condition.
c = Coefficient, determined by the perviousness of the surface of the water-shed.
R = Maximum rainfall, in 1 hour, in inches.
C′ = Coefficient.
The Value of C or the Maximum Run-off, in Second-feet per Square Mile, when R=1
Critical Time, in Minutes.

The following values, Table 8, result for this special case when water-sheds range in area from 10 to 10 000 sq. miles.

TABLE 8.—Relative Maximum Stream Flow from Similar Mountain Water-sheds of Various Extent.

In this table, R represents maximum rainfall in 1 hour.
(Special case, only, $K = 20$)

Area, in square miles.	Critical time, in minutes.	Maximum flow, in second-feet per square mile.	Total maximum flow, in second-feet.
10	63	302 R	3 000 R
20	89	250 R	4 880 R
50	142	188 R	9 440 R
100	200	149 R	14 700 R
200	283	122 R	24 400 R
300	346	108 R	32 400 R
400	400	99 R	39 600 R
500	450	92 R	46 000 R
1 000	630	75 R	74 000 R
5 000	1 410	44 R	220 000 R
10 000	2 000	34 R	340 000 R

The values in Table 8 show that in all mountainous regions exposed to the same rainfall conditions, the rate of maximum run-off, as determined by the formula, is about nine times greater per square mile from a water-shed having an area of 10 sq. miles and about four to five times greater from 100 sq. miles than from a water-shed having an area of 10 000 sq. miles. In the light of the stream-flow information obtainable from records of the U. S. Geological Survey, and other sources, and the conclusions reached in this matter by other investigators (Fig. 9), this relation appears to be quite reasonable. When account is taken of the fact that the value of R, that is, the maximum rainfall in 1 hour, may be much larger for those spots within the larger area at which the rain storms break with greatest severity, it will be readily seen that the disparities between the maximum run-off rate from the small area and that from the large area, as noted, which are predicated on the same value of R throughout the small area as throughout the larger area, may be considerably exceeded.

Other formulas for the approximation of maximum run-off under extreme conditions of rainfall are numerous. Most of these are intended to serve in restricted territory throughout which a similarity of rainfall conditions permits the use of run-off coefficients which are practically independent of the variations in the intensity of rainfall. Of this type, are the formulas suggested by the late Mr. Kuichling* for the New England and North Atlantic States; the Metcalf and Eddy formula;† the formula used by W. E. Fuller,‡ M. Am. Soc. C. E., and many others. Such formulas will naturally be used with some hesitation until they are supplemented with suitable correction factors to adapt them to meteorological conditions which depart materially

* "Report on the Barge Canal of New York," 1901, Part 14 of the "Report on Water Supply," by Emil Kuichling, p. 844.

† "American Sewerage Practice," by Metcalf and Eddy, p. 251.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

from those which obtain in the regions to which they are intended to apply. All such formulas, nevertheless, indicate in a more or less definite manner the conclusions of various investigators relative to the effect of topography and area on maximum stream flow.

The better type of formula is that in which the maximum stream flow is made a function of the intensity of rainfall as well as the area and topography and the soil or surface conditions of the water-shed. Most of these formulas contain a factor based on the precipitation conditions which will produce maximum run-off. Unfortunately, however, it is not always clear how this factor shall be determined. For comparison, the following formulas are briefly discussed.

A.—Formulas for Maximum Run-Off in Which the Intensity of Rainfall is a Factor.—The Bürkli-Ziegler formula:

$$D_m = C\,I\,M\sqrt[4]{\frac{S}{M}} \text{ sec-ft.} \quad (36)$$

and,

$$d_m = C\,I\sqrt[4]{\frac{S}{M}} \text{ sec-ft. per sq. mile.} \quad (37)$$

In this formula, I stands for the maximum intensity of rainfall in some definite time-period, preferably 1 hour. If some other time-period is selected, the effect will be to alter the coefficient, but not the ultimate result.

The McMath formula:

$$D_m = C\,I\,M\sqrt[5]{\frac{S}{M}} \text{ sec-ft.} \quad (38)$$

and

$$d_m = C\,I\sqrt[5]{\frac{S}{M}} \text{ sec-ft. per sq. mile.} \quad (39)$$

In this formula, too, I stands for the maximum intensity of rainfall in some definite time-period.

In this formula, and also in the Bürkli-Ziegler formula, the effect of the duration of the critical time-period, which lengthens as area increases, on the average rate of rainfall, and, therefore, on the rate of run-off, is not apparent as the factor, t, does not appear therein. Because this factor has been assumed to bear some more or less definite relation to the area and also to the slope of the water-shed, and as both area and slope appear in the formulas, the possibility that the formulas fairly approximate the most probable law of the variation of maximum run-off rate with area is not excluded.

B.—Formulas in Which Intensity of Rainfall Does not Appear as a Factor.—The Kuichling formulas:

(*a*) For drainage basins more than 100 sq. miles in area:

$$D_m = M\left(\frac{127\,000}{M + 370} + 7.4\right) \text{ sec-ft.} \quad (40)$$

and,

$$d'_m = \frac{127\,000}{M + 370} + 7.4 \text{ sec-ft. per sq. mile.} \quad (41)$$

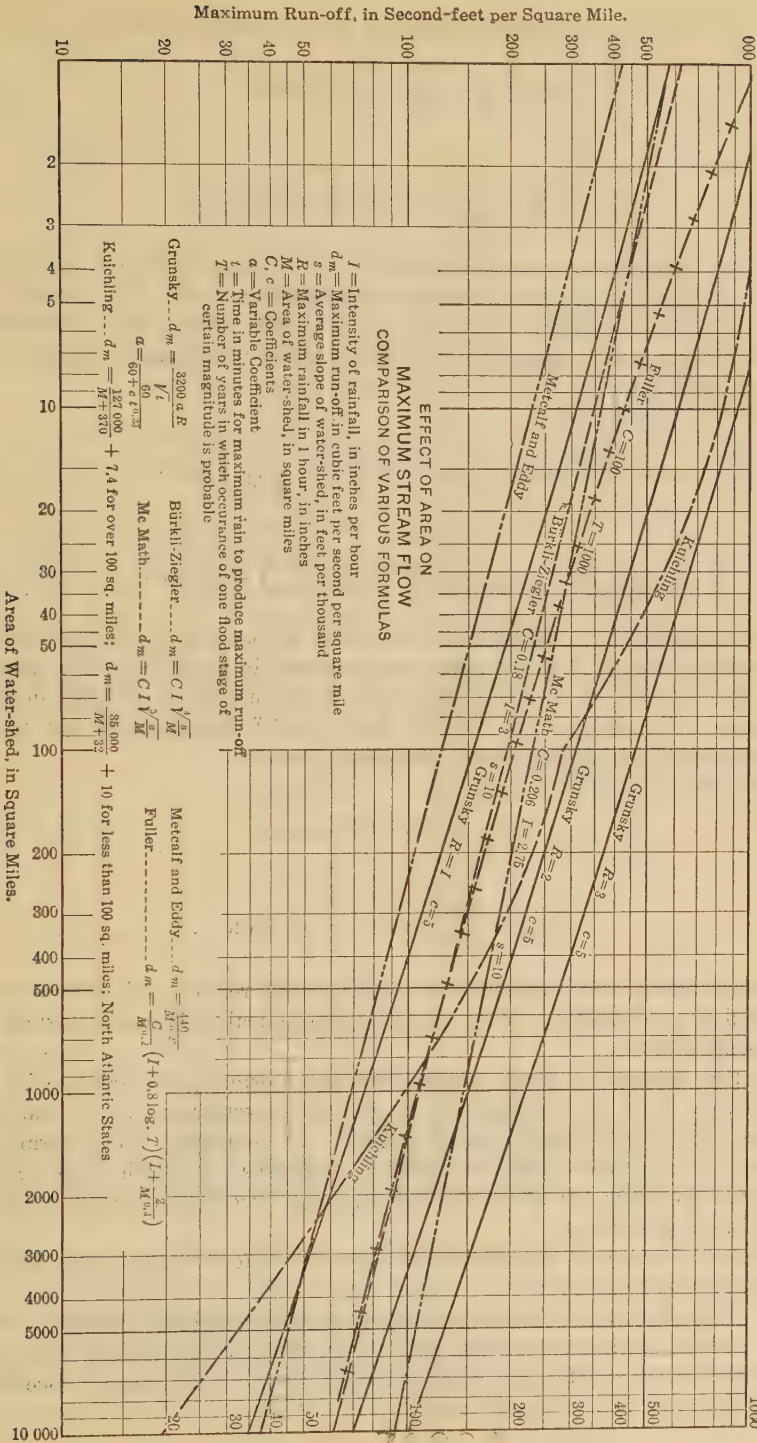

Maximum Run-off, in Second-feet per Square Mile.
EFFECT OF AREA ON
MAXIMUM STREAM FLOW
COMPARISON OF VARIOUS FORMULAS
I = Intensity of rainfall, in inches per hour
d_m = Maximum run-off in cubic feet per second per square mile
s = Average slope of water-shed, in feet per thousand
R = Maximum rainfall in 1 hour, in inches
M = Area of water-shed, in square miles
C, c = Coefficients
a = Variable Coefficient
t = Time in minutes for maximum rain to produce maximum run-off
T = Number of years in which occurance of one flood stage of certain magnitude is probable
Grunsky... $d_m = \frac{3200\ a\ R}{\sqrt{t}}$
$a = \frac{60}{60 + c\ t^{0.87}}$
Kuichling... $d_m = \frac{121\,000}{M + 370} + 7.4$ for over 100 sq. miles; $d_m = \frac{35\,000}{M + 32} + 10$ for less than 100 sq. miles; North Atlantic States
Birkli-Ziegler... $d_m = C\ I\ \sqrt[4]{\frac{s}{M}}$
Mc Math... $d_m = C\ I\ \sqrt[3]{\frac{s}{M}}$
Metcalf and Eddy... $d_m = \frac{140}{M^{0.7}}$
Fuller... $d_m = \frac{C}{M^{0.2}}\left(1 + 0.8 \log. T\right)\left(1 + \frac{2}{M^{0.1}}\right)$
Area of Water-shed, in Square Miles.
Fuller
C=100
T=1000
Kuichling
Metcalf and Eddy
Birkli-Ziegler
C=0.18
I=3
Mc Math
C=0.296
I=2.76
s=10
Grunsky
R=1
c=5
Grunsky
R=2
c=5
Grunsky
R=3
c=5
Kuichling

FIG. 9.

(*b*) For drainage basins less than 100 sq. miles in area:

$$D_m = M\left(\frac{35\ 000}{M + 32} + 10\right) \text{ sec-ft.} \quad \ldots\ldots(42)$$

and,

$$d'_m = \frac{35\ 000}{M + 32} + 10 \text{ sec-ft. per sq. mile.} \quad \ldots\ldots(43)$$

These formulas are intended to apply to hilly or mountainous regions and to meteorological conditions corresponding to those of the New England and the North Atlantic States. They are not applicable in regions in which rainfall conditions depart materially from those taken into consideration in shaping the formulas.

The Fuller formula:

$$D_m = C\,M^{0.8}\left(1 + 0.8 \log T'\right)\left(1 + \frac{2}{M^{0.3}}\right) \text{ sec-ft.} \quad \ldots\ldots(44)$$

and,

$$d'_m = \frac{C}{M^{0.2}}\left(1 + 0.8 \log T'\right)\left(1 + \frac{2}{M^{0.3}}\right) \text{ sec-ft. per sq. mile.} \quad \ldots\ldots(45)$$

In this formula, T' represents the number of years in which one storm of the intensity which is to be taken into account is probable. This formula, as in the case of all formulas in which the intensity of rainfall does not appear as a factor, has been deduced from run-off data without any attempt to interconnect rainfall intensity and rate of run-off.

The Metcalf and Eddy formula:

$$D_m = 440\ M^{0.73} \text{ sec-ft.} \quad \ldots\ldots(46)$$

and,

$$d'_m = \frac{440}{M^{0.27}} \text{ sec-ft. per sq. mile.} \quad \ldots\ldots(47)$$

This formula was suggested for a Kentucky region and should not be regarded as applicable elsewhere unless topographical and meteorological conditions are similar.

In Fig. 9, there is given in diagrammatic form a comparison of a few formulas to show the conclusions of various investigators as to the effect of area on the maximum rate of run-off. In preparing the diagram, the assumption was made that the rainfall conditions on which each curve is based are the same for all areas, although they were not assumed to be the same for all the curves.

The application of the maximum stream-flow formula can best be made clear by a few examples:

1.—What is the maximum discharge of a river draining a mountain watershed, 1 900 sq. miles in area throughout, on which the maximum rainfall in 24 hours is 8 in. and for which the critical time, $t = 720$ min.? (See Equation (25)).

In this case,

$$a = \frac{60}{60 + 5\sqrt[3]{720}} = 0.57$$

from Equation (3),

$$C = \frac{8}{24}\sqrt{1\,440} = 12.7$$

from Equation (1),

$$I = \frac{12.7}{\sqrt{720}} = 0.47$$

from Equation (25),

$$D_m = 413 \times 0.57 \times 0.47 \times 1\,900 = 210\,000 \text{ sec-ft.}$$

The conditions suggested in this example are comparable with those which prevail on the American River, above Folsom, Cal., which, from a water-shed of 1 900 sq. miles, at that point, has probably at its highest known stage discharged as much as 200 000 sec-ft. (1861-62).

2.—What is the maximum discharge of a river draining a water-shed three-fourths of which is mountainous and one-fourth rolling or foot-hill land, 9 000 sq. miles in area, in which the maximum rainfall in 2 days is 6 in. and for which $t = 1\,800$ min.?

In this example, the value of a will lie between the value determined for a mountainous area and that for a rolling country:

$$a = \tfrac{3}{4}\left(\frac{60}{60 + 5\sqrt[3]{1\,800}}\right) + \tfrac{1}{4}\left(\frac{60}{60 + 20\sqrt[3]{1\,800}}\right) = 0.42$$

From Equation (3),

$$C = \frac{6}{48}\sqrt{2\,880} = 6.7$$

from Equation (1),

$$I = \frac{6.7}{\sqrt{1\,800}} = 0.158$$

from Equation (25),

$$D_m = 413 \times 0.42 \times 0.158 \times 9\,000 = 247\,000 \text{ sec-ft.}$$

The conditions suggested in this example are comparable with those which prevail in the water-shed of the Sacramento River above Red Bluff, where the maximum recorded discharge from a drainage basin of 9 300 sq. miles has been about 254 000 sec-ft.

There is no need of extending illustrations to still larger water-sheds, because dependable basic data are not available and because the values of the constants and coefficients here introduced are only tentative. Enough has been said, however, to show that the type of the formula is reasonable. It will be found particularly helpful when from the known conditions of the flow from one water-shed, the maximum rate of run-off or stream flow from another with

similar meteorological and rainfall conditions, is to be estimated. It will also be useful in estimating maximum rates of run-off due to rains of extreme intensity when the maximum rates due to rains of less intensity have been ascertained by stream gauging.

When large parts of a water-shed are lake or reservoir surface, especial consideration may have to be given to the retarding effect of storage in the lakes or reservoirs. The retention of water in storage basins reduces the maximum rate of outflow therefrom and, therefore, reduces the peak of the discharge curve for points below the reservoir. Recourse may be necessary to the mass curve in order to determine this effect.*

Evaporation from Water Surface as Affecting Run-Off.

Whenever any considerable proportion of a water-shed is water surface, as when it embraces lakes of considerable extent, it will become necessary to give the water production due to a known quantity of rain especial study. The depletion of the water bodies or the accession of water which they have received from the beginning of one climatic year to the beginning of the next must be taken into account. This may involve a study of the loss of water by evaporation. When the rainfall is light, this loss of water from a lake by evaporation frequently exceeds the accession resulting from rain falling directly on the water. The water production of the lake area under such circumstances is negative. In computing the evaporation from the surface of a lake or reservoir, the following simple relation between mean monthly temperature and the probable average rate at which water is lost by evaporation throughout the month will be found to be helpful. This relation between the mean monthly temperature of the atmosphere and the rate of evaporation is shown in Table 9.

TABLE 9.—Evaporation from Open Water Surface.†

(Based on a revised curve.)

Mean monthly temperature, in degrees Fahrenheit.	Rate of evaporation, in feet per day.	Evaporation per Month:		
		In 28 days. Inches.	In 30 days. Inches.	In 31 days. Inches.
25	0.0015	0.50	0.54	0.56
30	0.0019	0.64	0.68	0.71
35	0.0025	0.84	0.90	0.93
40	0.0033	1.12	1.19	1.23
45	0.0043	1.48	1.55	1.60
50	0.0056	1.88	2.02	2.06
55	0.0073	2.45	2.63	2.71
60	0.0095	3.19	3.42	3.53
65	0.0121	4.06	4.32	4.50
70	0.0151	5.07	5.44	5.62
75	0.0186	6.25	6.70	6.92
80	0.0223	7.50	8.04	8.30
85	0.0260	8.74	9.37	9.68
90	0.0300	10.08	10.80	11.16

It seems to be fairly well established that evaporation increases somewhat with elevation above sea level. In the absence of any conclusive data relating

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 335.

† *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1968.

to the rate of this increase, the following formula is suggested for use until something better is offered:

$$E = E' + 0.0012 \sqrt{FH} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

In this formula, E is the monthly evaporation, in inches, at the place for which evaporation is to be determined, when the altitude of the place is H ft. above sea level; and E' is the monthly evaporation at sea level given in Table 9, which would obtain at the mean monthly temperature of F degrees, noted for any month at the place at which evaporation is to be estimated.

It must not be expected that for any single month evaporation will always be indicated correctly by the evaporation curve. Owing to the great variation in conditions of wind, humidity, sunshine, and the daily range of temperature, there may be wide departures from the monthly evaporation rate which the curve indicates as the probable rate. For 12 months, the probable error is much less than for any single month. The curve is intended for use when the annual evaporation is to be determined.

DISCUSSION

DANA M. WOOD,* M. AM. SOC. C. E. (by letter).—Engineers are still divided in their opinion regarding the advisability of publishing rainfall and run-off statistics for the calendar year or for some universally adopted climatic year. A committee of the Boston Society of Civil Engineers (the Run-Off Committee) sent out questionnaires to many engineers, asking their opinions on this question, and the replies were divided about equally. Many seemed to feel that particular studies in a specific locality might require the use of the climatic year, but opinion differed as to the best common climatic year to use, varying as it does in different parts of the United States.

The advantage in adhering to the calendar year for rainfall and run-off statistics appears to be threefold: (1) because of the non-uniformity of the climatic year; (2) it is the method used by the U. S. Weather Bureau in publishing the mass of rainfall figures already available, and the difficulties in bringing about a change, including a revision of past publications, would seem to be insurmountable for the present at least; and (3) the business reports of most municipalities and companies are on a calendar-year basis, and these reports often contain statistics of the nature discussed. The argument may sound like that of "locking the barn door after the horse is stolen" but, nevertheless, it is real. It is difficult to change long-established practice, even to substitute improved methods.

In regard to run-off records, engineers seem to be very definitely committed to the climatic year beginning October 1st, by the adoption of this system by the Water Resources Branch of the U. S. Geological Survey. The chief argument in favor of its general adoption seems to be that it permits of the continuous uninterrupted investigation of winter-flow conditions in those localities subject to ice conditions, and of the office preparation of report data at times of the year when field work is not so necessary; even this is not strictly true for all localities. The argument that it permits the issuance of published reports six months earlier than would otherwise be possible, does not seem to have so much force in recent years.

Summing up, it appears to matter little to the individual engineer which system is used because, in the majority of cases, he is forced to make his own transpositions and calculations on the basis of what in his judgment is the best division of the year for the locality he is studying. As long as the diversity of opinion exists, every one cannot be entirely satisfied, and the only satisfactory solution would be to use different divisions of the year for different sections of the country, and this also has its objections.

Referring to the discussion of "Rainfall in the Climatic Year", the question arises as to what is a normal rainfall. According to Weather Bureau data, the annual normal is the sum of the normals for all the months, and the normal for each month is the average for the period of record. Furthermore, normals are computed at stipulated intervals and are not always changed between times to obtain the correct figure to date.

* Engr., Stone and Webster, Incorporated, Boston, Mass.

Attention has been called to the value of the median.* In the paper, the study of frequency might be made theoretically more correct by the use of the median, but it is admitted that the final conclusions would be practically the same in this case, because there is so little departure between the two for the records used. Fig. 5 indicates this; but it is often not the case in other localities.

At the risk of repeating Mr. Grunsky's arguments, emphasis should be laid on the fact that, regardless of any arbitrary division of the year, the precipitation in the few months previous to the new year has a marked effect on ground-water levels and, therefore, on run-off. In other words, any method of analysis depending on the determination of the ratio of run-off to rainfall in any one given month will lead to wrong conclusions. Such ratios are of value for seasonal variations, but the length and dates of the several seasons will vary from year to year.

A fairly reliable estimate of the probable average, minimum, and maximum annual rainfall can be made for a given water-shed by the methods given in the paper. Other excellent papers indicate somewhat different methods for the same purpose.

In making estimates of the power available in a given water-shed, most engineers now desire both a hydrograph and a duration-of-flow curve, the latter including both the extreme maximum and minimum flows of record. If the annual run-off for the three typical years can be determined from rainfall statistics, the question of its relative distribution throughout the year is still to be determined.

So many different factors affect run-off that even with the same rainfall, in two different years, the run-off may be quite different both in quantity and distribution. The condition of the ground-water storage is an important factor, as well as other factors pointed out in the paper.

In order to determine the probabilities regarding run-off, some information which covers actual stream-flow measurements, even if for a short term of years, is imperative.

For a number of years the writer has intermittently been collecting and comparing run-off records compiled by a method similar to that used by Mr. Grunsky for rainfall records.

Duration curves of flow for the three types of years mentioned furnish a starting point. Discussing the average duration curve only in that which follows, the basis for comparison is always taken as the time scale, using it as percentage of total time for the period covered by the records and comparing flows for every 5% of the time. The flow is expressed as a percentage or ratio of the normal flow, the normal flow being taken as either the average or the median as may best suit the investigator's ideas on that subject.†

The value in using such a method lies in the comparisons afforded between what to the casual observer are entirely unlike streams. Records can be compared for widely different sizes of water-shed and for widely different annual run-offs; it is the relative distribution which is being studied.

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* *Engineering News-Record*, Vol. 80, p. 628.

† *Stone and Webster Journal*, February, 1917.

This naturally brings out the possibilities of classing streams as to their drainage-basin type, with or without varying degrees of storage development, etc. Of course, there are many gradual gradations from one class to another. This is true, also, for the classifications given by Mr. Grunsky, as it is necessary sometimes to adopt a value between those given.

TABLE 10.—Comparison of Merrimack and Connecticut River Records.

Percentage of time.	Ratio of Flow for Each Given Percentage of Time to Average.			
	October, 1880, to September, 1899.		October, 1900, to September, 1915.	
	Merrimack River at Lawrence, Mass. (4 570 sq. miles).	Connecticut River at Holyoke, Mass. (8 390 sq. miles).	Merrimack River at Lawrence, Mass. (4 452 sq. miles).	Connecticut River at Orford, N. H. (3 100 sq. miles).
100	0.005*	0.00*	0.008*	0.054
95	0.21	0.21	0.145	0.17
90	0.28	0.25	0.25	0.21
85	0.32	0.29	0.31	0.24
80	0.35	0.32	0.36	0.28
75	0.38	0.36	0.40	0.32
70	0.42	0.40	0.44	0.36
65	0.48	0.45	0.47	0.40
60	0.52	0.51	0.51	0.45
55	0.58	0.58	0.56	0.50
50	0.68	0.64	0.63	0.56
45	0.73	0.72	0.70	0.63
40	0.84	0.81	0.78	0.72
35	0.96	0.94	0.88	0.82
30	1.12	1.08	1.02	0.96
25	1.30	1.24	1.19	1.15
20	1.52	1.47	1.46	1.38
15	1.82	1.77	1.82	1.81
10	2.22	2.24	2.34	2.42
5	3.03	3.06	3.14	3.62
0	11.28	9.40	10.06	10.54
Average, cubic feet per second per square mile..	1.56	1.46	1.333	1.712

*Flow controlled by power plant.

In accordance with the writer's method, comparisons should be, as in rainfall studies, for the same period of years, but short-term records can be extended to long-term records just as readily as with rainfall records.

Studies along these lines have not been made to a sufficient extent to determine the full possibilities of the method and a classification system; but as an indication, Tables 10, 11, and 12 are given, covering several records analyzed in the foregoing manner. With such variations in actual stream-flow records in a restricted locality, as shown by these tables, the dangers of relying entirely on rainfall records are obvious.

Table 12 does not furnish an absolute comparison between records because the time-period is variable. It does show how several types of streams vary in run-off distribution. Extreme cases are selected.

The lengths of the records are as follows:

Column (2), 1878-1915 = 38 years, average flow = 0.538 cu. ft. per sec. per sq. mile.

Column (3), 1912-1916 = 5 years, average flow = 1.724 cu. ft. per sec. per sq. mile.

Column (4), October, 1905-August, 1917 = 12 years, average flow = 1.63 cu. ft. per sec. per sq. mile.

Column (5), October, 1903-September 1920 = 17 years, average flow = 2.19 cu. ft. per sec. per sq. mile.

Column (6), October, 1912-September, 1920 = 8 years, average flow = 1.61 cu. ft. per sec. per sq. mile.

Column (7), October, 1904-September, 1920 = 16 years, average flow = 1.68 cu. ft. per sec. per sq. mile.

Column (8), October, 1880-September, 1920 = 40 years, average flow = 1.51 cu. ft. per sec. per sq. mile.

Column (9), October, 1905-September, 1920 = 15 years, average flow = 1.63 cu. ft. per sec. per sq. mile.

Column (10), October, 1887-September, 1920 = 33 years, average flow = 1.50 cu. ft. per sec. per sq. mile.

TABLE 11.—Percentage of Time-Distribution of Flow Table, Maine Streams, 1913-20 (8 Years).

Percentage of time available.	Ratio of Individual to Average Flow.							
	St. John River at Van Buren. Av. 1.67. Drainage area = 8 270 sq. miles.	Machias River at Whitneyville. Av. 2.41. Drainage area = 465 sq. miles.	Union River, West Branch, at Amherst. Av. 2.18. Drainage area = 140 sq. miles.	Penobscot River at West Enfield. Av. 1.91. Drainage area = 6 600 sq. miles.	Piscataquis River near Foxcroft. Av. 2.65. Drainage area = 286 sq. miles.	Kennebec River at the Forks. Av. 1.76. Drainage area = 1 570 sq. miles.	Kennebec River at Waterville. Av. 1.86. Drainage area = 4 270 sq. miles.	Presumpscot River at Sebago Lake. Av. 1.50. Drainage area = 436 sq. miles.
100	0.06	0.009	0.04	0.12	†	0.08	0 013	†
95	0.13	0.18	0.10	0.32	0.06	0.31	0.25	0.31
90	0.16	0.23	0.14	0.37	0.09	0.39	0.34	0.40
85	0.19	0.28	0.19	0.40	0.12	0.45	0.40	0.53
80	0.22	0.31	0.22	0.42	0.14	0.52	0.44	0.69
75	0.26	0.35	0.27	0.45	0.19	0.58	0.47	0.81
70	0.31	0.39	0.32	0.47	0.23	0.64	0.50	0.86
65	0.36	0.45	0.38	0.51	0.28	0.69	0.52	0.92
60	0.41	0.51	0.46	0.54	0.33	0.75	0.54	0.96
55	0.46	0.57	0.54	0.58	0.41	0.82	0.56	0.99
50	0.52	0.63	0.62	0.63	0.50	0.88	0.58	1.02
45	0.59	0.70	0.72	0.70	0.58	0.93	0.62	1.04
40	0.67	0.79	0.84	0.77	0.68	0.99	0.67	1.06
35	0.76	0.94	0.97	0.88	0.77	1.06	0.77	1.09
30	0.91	1.23	1.11	1.00	0.85	1.13	0.92	1.13
25	1.10	1.32	1.33	1.15	1.03	1.20	1.14	1.18
20	1.39	1.56	1.58	1.39	1.37	1.27	1.44	1.21
15	1.85	1.85	1.93	1.77	1.86	1.38	1.81	1.24
10	2.68	2.32	2.44	2.24	2.62	1.56	2.32	1.28
5	4.03	2.98	3.33	3.15	3.84	2.24	3.22	1.36
0	8.76*	9.91*	6.65	7.04*	26.1	8.59	11.1	21.05

* Peak discharge.

† Controlled by plant above.

To study the monthly variations, it has been the writer's custom to prepare the duration-of-flow table so that duration-of-flow curves for each month can be also prepared. Obviously, the median for each month may be utilized in studying normal seasonal variations, and the duration curves indicate monthly extremes.

This general method can and has been used for obtaining a flow-duration curve where the run-off records are meager. Working from the rainfall to the estimated run-off for a given type of year, the distribution of this annual can be assumed in accordance with that in neighboring or similar basins for which there are reliable run-off records. If short-term records are available on the stream under investigation, the relative distribution for the period covered can be compared with that of other streams for the same period, and a neighboring record selected showing similar characteristics. From these data, an estimated curve for the longer period of years can be obtained, as described for the rainfall records.

TABLE 12.—PERCENTAGE OF TIME-DISTRIBUTION OF FLOW TABLE, MISCELLANEOUS STREAMS, VARYING PERIODS.

Percentage of time available.	Ratio of Individual to Average Flow.								
	Mississippi River at Keokuk, Iowa. Drainage area = 119 000 sq. miles.	Slippery Rock Creek at Wurtemburg, Pa. Drainage area = 400 sq. miles.	Kennebec River at Bingham, Me. Drainage area = 2 660 sq. miles.	Pemigewasset River at Plymouth, N. H. Drainage area = 615 sq. miles.	Housatonic River at Falls Village, Conn. Drainage area = 644 sq. miles	Connecticut River at Sunderland, Mass. Drainage area = 8 000 sq. miles.	Merrimack River at Lawrence, Mass. Drainage area = 4 452 sq. miles.	Androscoggin River at Errol Dam, Me. Drainage area = 1 095 sq. miles.	Presumpscot River at Sebago Lake, Me. Drainage area = 436 sq. miles.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
100	0.156	0.029	0.134	0.027	*	0.052*	*	0.045*	0.013*
95	0.28	0.042	0.25	0.16	0.135	0.17	0.19	0.44	0.24
90	0.32	0.063	0.30	0.19	0.18	0.21	0.27	0.52	0.41
85	0.36	0.088	0.35	0.22	0.22	0.25	0.32	0.58	0.53
80	0·40	0.11	0.40	0.25	0.28	0.29	0.36	0.63	0.65
75	0.44	0.15	0.44	0.27	0.33	0.32	0.39	0.67	0.77
70	0.49	0.19	0.48	0.30	0.37	0.36	0.42	0.72	0.82
65	0.54	0.26	0.52	0.33	0.41	0.41	0.45	0.77	0.86
60	0.61	0.33	0.56	0.365	0.47	0.46	0.51	0.81	0.93
55	0.67	0.42	0.62	0.40	0.55	0.52	0.57	0.86	0.98
50	0.75	0.48	0.67	0.45	0.62	0.58	0.63	0.90	1.02
45	0.83	0.53	0.73	0.52	0.74	0.67	0.71	0.94	1.04
40	0.94	0.64	0.78	0.61	0.84	0.77	0.80	0.98	1.08
35	1.05	0.75	0.86	0.72	0.97	0.89	0.92	1.02	1.12
30	1.18	0.90	0.97	0.88	1.13	1.05	1.13	1.06	1.16
25	1.35	1.08	1.10	1.09	1.29	1.27	1.25	1.10	1.21
20	1.55	1.64	1.33	1.44	1.50	1·54	1.49	1.16	1.26
15	1.78	1.76	1.67	1.90	1.88	1.90	1.79	1.25	1.34
10	2.05	2.31	2.22	2.52	2.40	2.44	2.22	1.46	1.43
5	2.49	3.52	3.10	3.91	3.07	3.28	3.00	2.22	1.55
0	4.9	25.8	8.7	13.9	8.5	8.0	11.9	7.0	21.0

* Controlled flow.

Maximum and minimum probable daily run-off estimates can be made along the lines suggested, but such results should always be checked against determinations made by other methods and the most reasonable value adopted, unless reliable long-term flow records are available near-by.

The purpose of this discussion has been to offer additional suggestions which may prove helpful in what is at best a very difficult and uncertain problem. Changes in run-off régime, as in rainfall, are abrupt, and result from many factors operating some against, some with, each other. In the last analysis, only general tendencies or types of years can be predicted and, even after the most intensive study, departure from the expected will be encountered.

In conclusion, the writer feels that even with the relatively short-term run-off records available in the West, streams with long-term records can be found elsewhere, which will furnish a basis for extending the short-term record. Results obtained in this manner should furnish an excellent checks on determinations made entirely from rainfall data.

C. F. Marvin,* Esq. (by letter).—On behalf of the U. S. Weather Bureau the writer may state that Mr. Grunsky's request for a change in the annual publication of meteorological data, in that the annual summaries should apply, throughout the whole country, to a climatic year and not to a calendar year, was received and considered in a sympathetic manner. Shortly after its receipt, letters requesting expressions of views on the subject-matter therein were sent to various persons and organizations. The response has not been as full and satisfactory as could be desired, and, therefore, a second inquiry and questionnaire is being distributed to recipients of the *Monthly Weather Review,* in the hope that discussion on this important proposition will be stimulated.

As Mr. Grunsky's request appears to have been based mostly on the needs of students of rainfall in California, it has been thought desirable to secure a wide expression of views, and, therefore, correspondence and discussion on the subject, on the part of engineers, are solicited by the Weather Bureau.

Rudolph Hering,† M. Am. Soc. C. E.—The question of rainfall and run-off studies has been before engineers for a great many years. About forty years ago, when the speaker had to face the problem for the first time seriously, it seemed almost hopeless to obtain what an engineer likes—exactness—some formula on which one could positively rely. About that time, the speaker was also confronted with the use of formulas for similar physical data as aids in the solution of problems, and it sometimes seemed hopeless to use formulas for any series of occurrences.

This remark was once made about some pile-driving experience to the editor of *Engineering News,* who requested something about that particular subject for publication, which information was subsequently furnished him. This was in regard to the Fairmount Bridge in Philadelphia, Pa. There was trouble about the piling, and the question arose as to whether or not the engineer could be sure that the pile was going to hold up the weight to be placed on it when driven with a certain weight of hammer and a certain distance of fall. The speaker began to collect all the formulas available in America, England, France, and Germany on the subject, with the thought of finding out what those various formulas really meant. A heavy and a light hammer and a high and a low fall were assumed and the results were worked out. These results were published in *Engineering News* (1889), and the late Arthur M. Wellington, M. Am. Soc. C. E., one of the editors, deduced afterward from those formulas the one now known as the Wellington formula.

Gen. McAlpine deduced the formula known as the McAlpine formula from experiments made in the Brooklyn Navy Yard. He gave it to the world

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† Cons. Engr., New York City.

without any limitations, and it was used by engineers. It will be seen from that article in *Engineering News,* assuming a possible case of pile-driving, that the more the pile was pounded the less it would carry; in other words, the result was negative.

Now, of what use is such a formula? In fact, many formulas are questionable. The speaker has suggested that every one offering a formula should do so as in calculus, when the two limits to the integral sign are given, meaning that this formula is applicable within those limits and is no good outside of them.

One reason why Kutter's formula interested the speaker was because it comprised all cases from a 1-in. pipe to the Mississippi River. All the gaugings used were between those two limits, and the formula works astonishingly well under all circumstances if the coefficient of roughness is judged correctly.

This shows that the use of a formula by a practical engineer is somewhat dangerous, unless he knows the data from which the formula has been derived. In that case, of course, engineers have no better guide than a formula, because it practically gives them a curve between known facts, and is safe to use; beyond that, it is not safe.

Another case was published about 1878.* There was a discussion on the same subject as that presented by the author, and a formula for run-off was given. It was shown in the discussion how easy it was to obtain absurd results; and the speaker presented another method of getting those results, namely, by plotting curves from actual gaugings under different conditions. It was felt that those limited curves which were established from known facts, would always be reliable, whereas a formula representing a curve of perhaps infinite length, might lead to great error. Data had been collected of the run-off for many streams in America, Europe, and India, where heavy rain storms and floods occur, giving the maximum flood discharges down to small areas, such as the municipal engineer finds in sewerage problems.

Engineers should feel grateful to Mr. Grunsky for having compiled so many data and made such a thorough study of a very important subject which in the collection of data has by no means as yet been exhausted. He has given sufficient information to permit an engineer to proceed with more confidence in the results than without it.

In the Philadelphia Water Department, from 1883 to 1886, some experiments were made with some reference to this subject. The streams were gauged with automatically registering gauges. Not only was the total rainfall measured, but there was also automatic continuous registration. After the speaker left that Department, 36 years ago, these observations were continued, and if the data were properly compiled, they would furnish an unusual amount of information of the run-offs from areas of different topography varying from 100 to 400 sq. miles, extending over a period of from 25 to 30 years. The areas differed in their physical character, some being hilly and some flat; some were much wooded, and others were open agricultural districts.

In order to be able to interpret intelligently these run-offs, with the permission of the Chief Engineer of the Department, the late William Ludlow,

* *Proceedings,* Engrs. Club of Philadelphia, Vol. 1 (1880), p. 146 *et seq.*

M. Am. Soc. C. E., a few acres of land—about half a dozen—with different physical characteristics were staked off. For instance, one was very steep and wooded, another steep but plowed agricultural land; then, again, there was a very flat area wooded and one that was open.

The rain was measured in a centrally placed gauge, and the run-off was measured by a water meter placed at the foot of the area, so that it made an interesting and valuable comparison. The results were never published, and the speaker does not know what became of them, but the original data must still be in the files of the Philadelphia Water Department.

Surveys were made of more than 400 sq. miles, and all the wooded and open areas, roads, and even every house in the area, were located, so that its physical character could be determined very closely. Topography over the whole area was taken with 10-ft. contour intervals. Therefore, data for a very interesting study of run-off may be found there recorded.

Olin H. Landreth,* M. Am. Soc. C. E.—The membership of the Society, and the Profession at large, are under great obligations to Mr. Grunsky for his repeated valuable contributions to the technical literature on hydrology, including this interesting and valuable undertaking to derive empirical rules relating to rainfall and run-off on the Pacific Slope.

In referring to the fact that the ratio of run-off to rainfall is greater on high mountainous areas than at low altitudes, the author attributes the difference, not to "the character of the surface of the water-shed", but rather to the smaller loss from evaporation in high mountainous areas, due to the lower temperature, and, consequently, the greater proportion of rainfall remaining as run-off.

The fact is not questioned that a greater ratio of run-off to rainfall prevails in high altitudes than in low altitudes, nor is it doubted that the smaller loss from evaporation in the higher altitudes constitutes the main, if not the only, cause of this difference; but, perhaps, it may be questioned whether the lower temperature in mountainous altitudes is the only cause for the reduced loss from evaporation.

In general, evaporation per unit of superficial area from the free surface of water and, at much lower rates, from the surface of snow and ice, mainly depends on, and varies with, three elements: (1) the temperature of the evaporating surface or film; (2) the humidity and the degree of agitation of the air above the evaporation surface; and (3) the length of time of exposure to evaporation. If, instead of a free water surface, the evaporation is from the surface of soil supplied with moisture from ground-water, an additional factor also affects the evaporation rate, namely, (4) the rate at which water is drawn up to the ground surface by soil transpiration, which, in turn, depends on the minimum transpiration capacity of the soil for a unit of depth, and also on the depth of the ground-water below the surface of the soil.

At high mountainous altitudes not only are temperatures generally lower than on the lower plain areas, but the average time of exposure to evapora-

* New York City.

tion is also less, because of the average steeper slopes and resulting higher velocities, both for the free running water and for the ground-water moving down the slopes by percolation.

The average time of exposure to evaporation should vary approximately inversely as the square root of the slope. Thus, with other controlling elements the same, but with comparative slopes of 2° and 8°, respectively, the latter should give average times of exposure to evaporation about one-half those of the former, and as evaporation varies directly with the time of exposure, the evaporation should be about one-half as much on the steeper as on the flatter slope. Also, the generally greater coarseness of soil grains on mountainous areas, as compared with flat areas, favors a more rapid rate of percolation of ground-water and, therefore, still further tends to shorter times of exposure and consequent smaller evaporation loss.

In addition, even with equal temperatures, the evaporation loss from the ground surface should be less on mountainous than on lower plain areas, due to the lesser capillarity and, therefore, lesser transpiration capacity of the coarser mountain soils, as compared with the finer grained soils of the lowlands.

From the foregoing, it would appear that the generally lower temperatures in mountainous areas should not be assumed to be the only cause of the observed lower evaporation losses, but rather that the differences in average slopes and in average soil fineness should be recognized as playing parts—and, perhaps, important parts—in causing the smaller evaporation losses in mountainous as compared with low-level areas.

If this is true, then the desirability should evidently be considered of modifying Equations (13) and (14), which are offered by the author as means of predicting the run-off in mountainous regions from that at low altitudes. These equations at present contain only one factor to account for the difference in run-off, namely, f, the mean temperature throughout the water-shed in question during the "wet season" from December to May.

If, however, it should be decided not to modify the present form of Equations (13) and (14), then, notwithstanding the factor, f, is still retained and used in its defined meaning, the empirical coefficient, C, when evaluated from a sufficiently wide range of observations on the rainfall, P, and on r and r', the run-off from the low-level and mountainous areas, respectively, will inevitably contain the effect of all the elements which really cause the observed difference in the ratio of run-off to rainfall in the high and low areas, respectively, even though no terms are introduced into the equations to represent such other elements than temperature.

Thaddeus Merriman,* M. Am. Soc. C. E.—The papers presented by Mr. Grunsky and by Messrs. Munn and Savage, illustrate two of the prevailing methods used in the study of flood magnitudes. In the first method a formula is deduced to express the maximum flood discharge in its relation to the intensity of the rainfall and the physical features of the water-shed. The second, premising an observed maximum flood discharge, recognizes the possibility of the occurrence of a still greater flood and increases the observed values in some

* Chf. Engr., Board of Water Supply, New York City.

definite proportion, as by assuming an increase in the total run-off and then working backward for the purpose of determining the probable peak discharge. The first of these lines of thought has been well set forth by Mr. Grunsky and the second by Messrs. Munn and Savage. A third method which is perhaps the one most commonly used, consists in plotting observed maximum flood discharges with respect to the area of the drainage basins on which they occurred. An expression is then determined which will enclose, and yet fairly represent, the plotted points.

Each of these three methods constitutes a perfectly legitimate way of approximating the truth, but only the second stresses the fact, which the speaker considers to be of great importance, that greater floods than any which have been observed may occur at any time. No formula or expression for flood discharge can be used implicitly as it is written. Allowance always must be made for the case which the formula does not include.

The one outstanding feature in any tabulation of recorded floods seems to be that most of them have occurred in recent years. In a tabulation which lists 274 floods in the United States, 219 have occurred since 1890, and evidently most of them are far from being possible maxima. Obviously, this does not mean that more floods have actually occurred in recent years, but is simply the result of the absence of long-time records.

In 1914, the speaker visited the Ohio region which had been desolated by the floods of March in that year. He then became much interested in this question of probable maximum flood magnitude and has given it a great deal of thought and study. This matter is one of great moment. Our cities, day by day, are spreading wider and farther over the flood-plains of the rivers on the banks of which they were originally founded. The buildings, bridges, fences, and other structures operate to obstruct the free discharge of the flood waters and, where the flood-plain is covered, the greatest damage occurs along the line of greatest resistance. This line of greatest resistance, unfortunately, often lies within the city and accounts for the appalling devastation which has come to be a rather common occurrence. Immediately after such a flood, the newspapers carry large headlines describing it as "unprecedented", "of a magnitude never before known" and as "a wall of water which swept everything before it". After a brief lapse, when the toll of the dead and the missing has been taken, the occurrence is promptly forgotten. Only in those cities which were the actual sufferers do the people live in the fear of another similar occurrence.

It is not necessary to look far to note that practically all sections of the United States are subject to a flood hazard. The records of the very recent past clearly demonstrate this and within the past 20 years "unprecedented" floods have occurred in New Jersey, New York, Pennsylvania, Ohio, Indiana, Iowa, North Carolina, Tennessee, South Carolina, Texas, Colorado, Arizona, and perhaps, the most remarkable of all in regard to volume, occurred at Monterey, Mexico. The list might be extended, but emphasis would not be gained thereby.

Each new flood is "unprecedented" simply because the records are meager and cover a period too short to be of great value. The importance of a flood in

this day and generation is measured by the destruction which it occasions, whereas a flood of equal magnitude, fifty or seventy years ago, might easily have passed unnoticed and unrecorded. When one of these "unprecedented" floods does occur, the newspapers and subsequent technical reports record the failure of one or more dams due to inadequate spillway capacity. Most dams are designed with a factor of safety of about two, but very few spillways are designed with any factor of safety; in many instances, they are made merely large enough to pass the greatest flood of record in the particular vicinity of the dam, or are proportioned on the basis of a formula and no allowance is made for the occurrence of a flood greater than those on which the formula was based.

In view of the generally unsatisfactory knowledge respecting flood magnitudes, the speaker desires to urge the necessity of providing ample spillway capacity on all dams, wherever they may be built, and as a guide to the possible maximum flood, that greater dependence be placed on the observable physical features of the flood-plains of the river itself than on any records of flood discharge which are now available or will become available for many years. By the observable physical features of the flood-plains of the river itself, the speaker refers to those evidences which a flood always leaves. He does not, however, allude to the ordinary high-water drift of logs, etc., which is ephemeral, but refers to those relatively permanent and definite evidences in the deposits of sand and gravel found on the flood-plains near the points where tributary streams first emerge on the flood-plain of the main river valley. When a river is in flood and its flood-plain is covered, the tributary streams flowing at a relatively high velocity are suddenly checked when they enter the comparatively quiet water over the flood-plain. At these points their burden of sand and gravel is deposited in the form of flat topped banks or terraces, the tops of which are only a little below the high-water level in the main stream. Later, these banks may be more or less eroded as the flood subsides and the tributary flows over and scours out the material previously deposited. A study of these evidences is of the nature of a geological investigation with respect to the recent valley deposits and will disclose the heights at which flood waters in the valley have stood at some time in the past. In this manner, the records can be carried back farther than by any other method of which the speaker is aware. He has seen gravel banks of this nature on the Miami River, which were deposited during the flood of 1914, and the tops of these banks are definite and permanent marks of the height at which the flood waters stood in that year. He also has seen similar deposits in North Carolina at elevations above those reached by the great flood of 1916, which flood at certain points raised the previous maximum stage from 21 to more than 42 ft.

Those who are located within the area which was covered with ice during the glacial period have relatively few opportunities for making such studies. In these glaciated areas, it is generally very difficult to distinguish clearly between the deposits of the glacial age and those of comparatively recent formation. Therefore, the suggestion is made that other members of the Society, located in unglaciated areas, by a study of these features, may be able to add greatly to the present imperfect knowledge respecting probable maximum flood heights.

R. A. Hill,* Assoc. M. Am. Soc. C. E. (by letter).—A method somewhat similar to that described by Mr. Grunsky was developed by the writer to determine the probable rainfall on the mountains between the Southern California valleys and the deserts which lie back of them. The topography of that section of California south of the Tehachipi Mountains is characterized by high ranges which roughly parallel the coast at a distance back of from 40 to 80 miles and which rise out of a plain generally less than 1 000 ft. above sea level. Several of the peaks are more than 10 000 ft. in elevation, although the crests of these ranges are, in general, at elevations of from 5 000 to 8 000 ft. Twenty miles across the mountains from the highly developed coastal valleys lie the Mojave and Colorado Deserts.

The average rainfall per season, in the coastal valleys, is about 16 in. On the ocean side of the mountain ranges, the precipitation increases very rapidly for increases in elevation, up to about 5 000 ft. Above that point, the rate of increase is small, and, as pointed out by the author, there is an apparent decrease for the last few hundred feet below the crest. The rainfall, including snowfall, at or near the crest, is approximately twice as great as in the valleys a few miles below. On the desert side of the crest, the rainfall decreases rapidly and at the same elevation as the coastal valleys, only about one-quarter as much rain falls in any season.

The direction of the storms is usually from the southwest, off the Pacific Ocean. The irregular crest line and the high peaks close to the ocean side cause very definite rain shadows.

An excellent example of this is found near the top of the San Bernardino Mountains in the vicinity of Big Bear and Baldwin Lakes. The precipitation averages about 32 in. per season at the dam at the west end of Big Bear Lake (Elevation 6 500), whereas 5 miles distant at the east end of the lake, the rainfall is about 21 in. per season. At Baldwin Lake, which lies about 4 miles still farther east at about the same elevation, the rainfall is about 16 in. per season. This rain shadow is caused by the high peaks of the San Bernardino and San Gorgonio Mountains and the connecting ridge, all of which are over an elevation of 10 000 ft. and intercept the storms from the ocean. The west end of Big Bear Lake, however, is clear of the shadow of these higher mountains.

The scarcity and the relatively short duration of records of precipitation gave rise to a method quite similar to that described by the author. Instead of determining the mean for the period available and comparing that with the same period at the base station to ascertain the ratio between the rainfalls at the primary and secondary stations, this ratio was determined for each season. The long record of the rainfall at San Bernardino (1870 to date) and the geographic position of that city made it desirable to refer all other stations to it.

Accordingly, with the rainfall each season at San Bernardino as a base, the rainfall for the same season at all other available stations along and back of these mountain ranges was computed as a percentage of that at San Bernardino. These points, several hundred in number, were then plotted with

* Engr., Quinton, Code and Hill, Los Angeles, Calif.

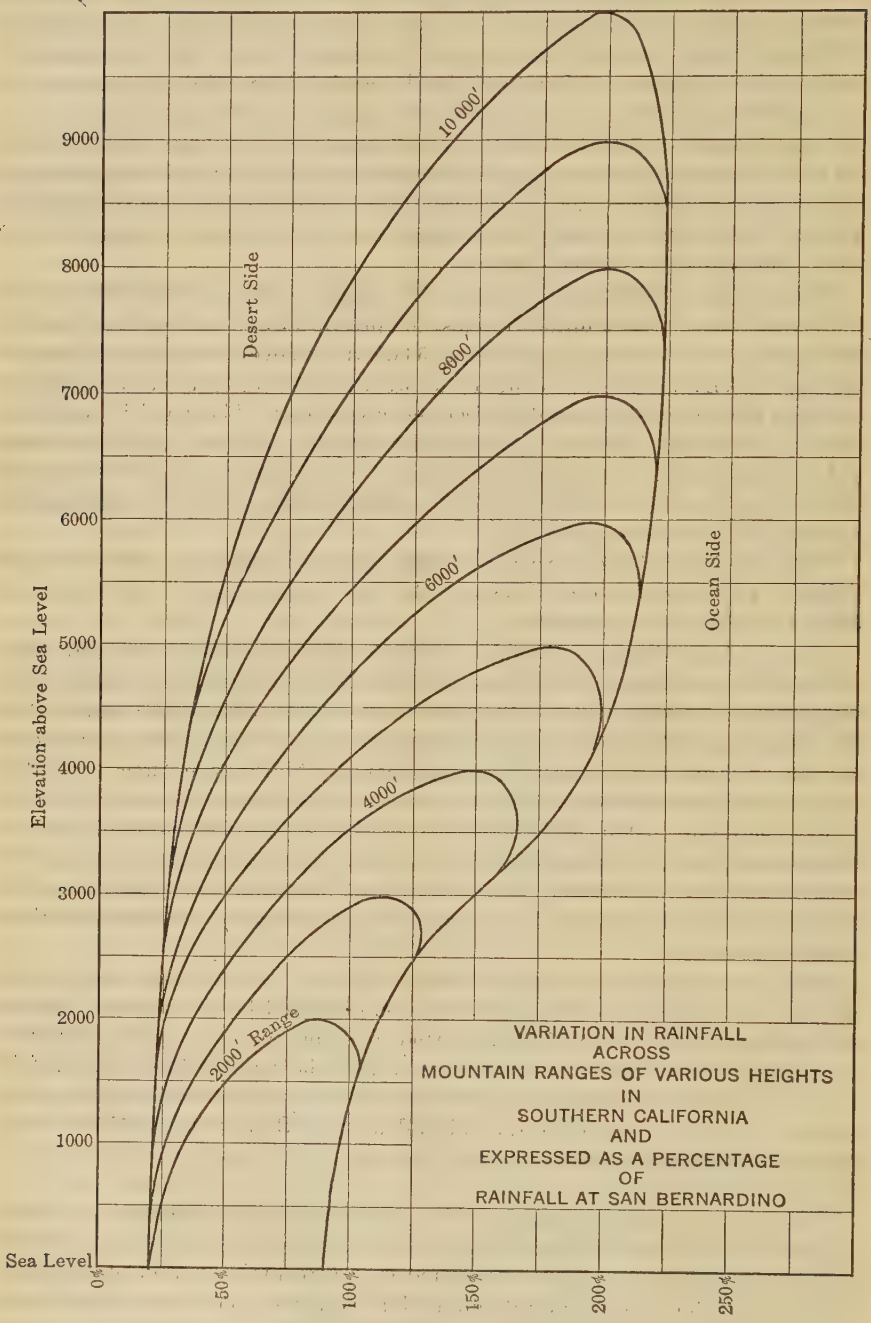
Elevation above Sea Level
9000
8000
7000
6000
5000
4000
3000
2000
1000
Sea Level
Desert Side
Ocean Side
10 000′
8000′
6000′
4000′
2000′ Range
0%
50%
100%
150%
200%
250%
VARIATION IN RAINFALL
ACROSS
MOUNTAIN RANGES OF VARIOUS HEIGHTS
IN
SOUTHERN CALIFORNIA
AND
EXPRESSED AS A PERCENTAGE
OF
RAINFALL AT SAN BERNARDINO

Fig. 10.

elevations as ordinates and percentages of the rainfall at San Bernardino as abscissas. There was considerable fluctuation in the points, as was to be expected from the widely variant conditions which exist over these mountains. Curves drawn through the weighted centers of these points gave much the same result as if the ratio between the means had been taken in the first place.

The resulting wing-shaped diagram (Fig. 10), fits quite satisfactorily the conditions known to exist on the ocean and desert sides of the mountains of Southern California. Table 13 gives the results of such comparisons for some

TABLE 13.—RAINFALL OF SOUTHERN CALIFORNIA STATIONS AS COMPARED TO SAN BERNARDINO.

Station.	Elevation, in feet.	Position relative to mountains.	Elevation at crest of mountains, in feet.	Probable percentage of rainfall at San Bernardino.	Mean rainfall for same period at San Bernardino, in inches.	Probable mean rainfall at station, in inches.	Actual mean rainfall at station, in inches.	Difference, in inches.
Barstow	2 105	Desert	8 000	25	14.98	3.74	3.62	+ 0.12
Cabazon	1 779	Desert	2 500	55	15.09	8.30	10.48	− 2.18
Mojave	2 751	Desert	6 000	35	16.14	5.65	5.00	+ 0.65
Palm Spring	584	Desert	10 000	20	15.42	3.09	4.13	− 1.04
Tehachipi	3 964	Desert	5 500	75	16.14	12.10	10.61	+ 1.49
Mt. Wilson	5 850	Crest	5 850	195	14.43	28.20	25.20	+ 3.00
Bear Valley Dam	6 500	Ocean	8 000	220	14.71	32.40	32.74	− 0.34
Glen Ranch	3 258	Ocean	9 000	165	16.68	27.50	30.44	− 2.94
Lowe Observatory	3 240	Ocean	5 000	165	14.75	24.35	25.18	− 0.82
Mill Creek	2 950	Ocean	10 000	145	16.83	24.40	24.50	− 0.10
Squirrel Inn	5 280	Ocean	6 000	215	17.96	38.60	37.32	+ 1.28

of the typical stations used in developing the diagram. Therefore, it can be assumed, inductively, that the use of this diagram would give reasonable values for the rainfall across these mountain ranges of varying crest height. Several reservoirs for power and irrigation have been constructed on top of these mountains, and in the determination of the feasibility of others projected, this diagram has been of considerable value as an index of the probable water supply from drainage areas with these variable characteristics as to precipitation conditions.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—The author's comprehensive survey of the whole question of rainfall and run-off is a valuable contribution to the already voluminous literature on the subject. Relative to sewer design, to which this discussion is confined, a lamentable lack of uniformity in practice still exists, particularly with regard to run-off, an indication that certain elements in the computations are interpreted differently or are accepted with hesitation in the absence of complete fundamental data.

For an intensity formula, the expression, $I = \frac{C}{t^{\frac{1}{2}}}$, is recommended, and there is much to be said in its favor because of its simplicity and its fair accordance with observed results; but the writer believes that where long-time gaugings are available, modifications of this formula will often show a closer agreement with the curve plotted from gaugings.

The long series of self-registered gaugings at Central Park, New York City, covering 51 years, give an admirable opportunity for arriving at the

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York City.

probable frequency of different intensities, and the writer has attempted this by a method which is described in *Engineering News-Record* of April 7th, 1921.* Approximate formulas for the resulting curves shown in Fig. 11 are as follows:†

$$\text{5-year curve} \ldots\ldots I = \frac{25}{(t+4)^{0.67}}$$

$$\text{10-} \quad \text{“} \quad \text{“} \quad \ldots\ldots I = \frac{43}{(t+8)^{0.75}}$$

$$\text{15-} \quad \text{“} \quad \text{“} \quad \ldots\ldots I = \frac{47}{(t+8)^{0.75}}$$

$$\text{25-} \quad \text{“} \quad \text{“} \quad \ldots\ldots I = \frac{78}{(t+11)^{0.84}}$$

$$\text{50-} \quad \text{“} \quad \text{“} \quad \ldots\ldots I = \frac{186}{(t+17)}$$

$$\text{100-} \quad \text{“} \quad \text{“} \quad \ldots\ldots I = \frac{354}{(t+24)^{1.1}}$$

$$\text{Absolute maximum curve} \ldots I = \frac{74\,000}{(t+80)^2}$$

In Table 1, the author gives a comparison between his proposed “parabolic formula” and the “New York formula”.‡ Table 14 is a comparison of these formulas with those determined by the foregoing formula for 10 and 15-year frequencies, and is confined for the most part to values of t used in sewer computations.

TABLE 14.—Rain Intensity, in Inches per Hour.

t.	Brooklyn formula.	Parabolic formula.	10-year New York formula.	15-year New York formula.
5	6.0	6.5	6.28	6.86
10	5.0	4.6	4.92	5.38
20	3.75	3.24	3.53	3.86
30	3.00	2.65	2.81	3.07
60	1.88	1.88	1.82	1.98
120	1.07	1.32	1.13	1.24
720	0.20	0.54	0.31	0.34

The writer believes that the coefficients for run-off in general use should not receive the confidence which they are accorded, and has been unable to discover

* Also referred to on p. 95.

† Suggested by Prof. Edward S. Allen, State College, Ames, Iowa, who sends the following more precise formulas for the 10 and 15-year curves:

$$\text{10-year} \ldots I = 27\, t^{-\frac{3}{5}}\, 20\, e^{-\frac{3}{5}t}\, t^{-\frac{4}{5}}$$

$$\text{15-year} \ldots I = 30\, t^{-\frac{3}{5}} - 32\, e^{-0.1\,t}\, t^{-1.1}$$

‡ The author's use of the terms, “New York Formula”, for $I = \frac{150}{t+20}$, is not strictly correct. Heretofore, each of the five Boroughs of the city has used its individual formula, and the foregoing formula adopted by the Borough of Brooklyn many years ago, is used by that Borough only. After studies resulting in a report of the Committee on Rainfall and Run-off of the Municipal Engineers of the City of New York and published in its *Proceedings*, for 1913, the Borough of Manhattan adopted the 10-year formula there proposed, namely, $I = \frac{150}{t+16}$; but, in its application, t is taken to exclude the 4-min. inlet time which is added to the 16 min., giving it the form of the Brooklyn formula, but with a different value for t. In this manner, the two Boroughs were inadvertably mentioned by the writer as using the same formula in his discussion of the paper by L. Standish Hall, Assoc. M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 232.

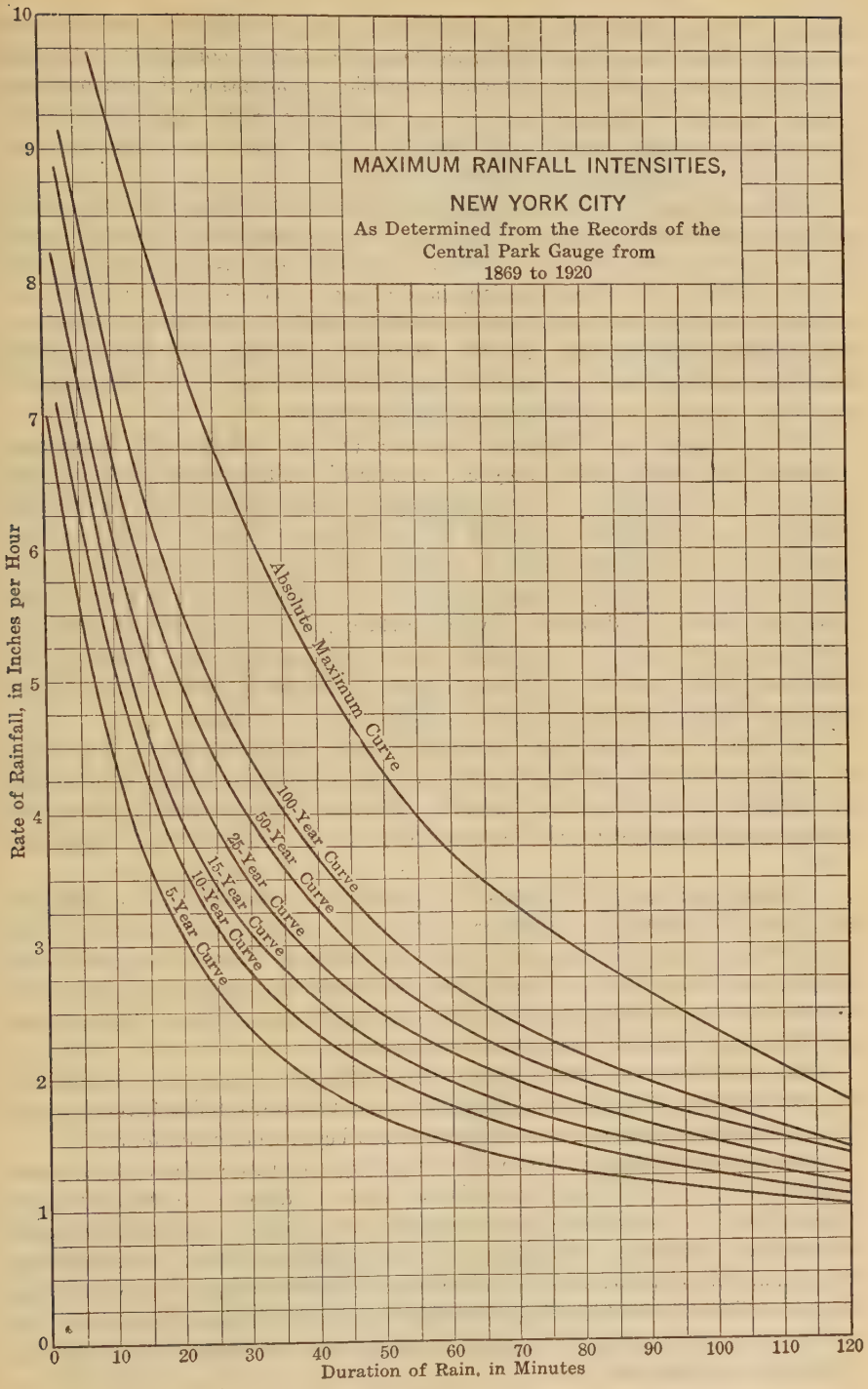
MAXIMUM RAINFALL INTENSITIES,
NEW YORK CITY
As Determined from the Records of the
Central Park Gauge from
1869 to 1920
Absolute Maximum Curve
100-Year Curve
50-Year Curve
25-Year Curve
15-Year Curve
10-Year Curve
5-Year Curve
Rate of Rainfall, in Inches per Hour
Duration of Rain, in Minutes
0
1
2
3
4
5
6
7
8
9
10
0
10
20
30
40
50
60
70
80
90
100
110
120

FIG. 11.

many recorded experiments for determining the percentage of run-off from areas of a single character. Although there is a general consensus of opinion among engineers as to run-off factors suitable for adoption, it is a question whether these have not been adopted from precedent rather than from knowledge. Mr. Grunsky has well said that "the effect of this perviousness must be made by the engineer, on the basis of experience."

Of the various elements entering into the determination of R from which erroneous results may be derived, the time of concentration and the gauging of the flow in the sewer are all important. Reliable information regarding the velocity of flow over different surfaces and along gutters is lamentably lacking, so that in practice the inlet time assumed is largely a matter of conjecture. Under urban conditions the error due to this is not often serious but, unless great care is taken, there are possibilities of serious errors in the computations of flow in the sewer, as any competent engineer knows.

Weirs are costly and difficult to introduce and the collection of débris may invalidate the results unless they are constantly under supervision; whereas dependence on computation by a formula may lead to serious error by varying hydraulic conditions, the selection of an improper friction coefficient, the assumption of the grades shown on construction plans instead of those which exist, and the complications introduced by sludge deposits, melting snow, bends, junctions, volume of domestic sewage, surface slope, the hydraulic effect of conditions above and below the section under consideration, etc. The fact that more than 100% run-off is frequently shown in tabulated results is an indication that errors of the nature mentioned have been introduced.

The writer has recently had occasion to study the proper run-off factor for use under completely developed urban conditions and has made inquiry into the practice of various municipalities. Among those questioned there was quite a general agreement with percentages ranging roughly from 75 to 90 per cent.

Attention has been called recently to a number of densely built-up districts in a large city where the capacities of the main trunk sewers are only from 33 to 44% of those required for a 100% run-off and where no complaints of flooding have ever been received. This, of course, does not mean that these sewers were never surcharged, but the question at once arises: Why provide for 80% run-off if 40% answers every purpose?

The writer does not wish to appear as an advocate of the lower coefficients, but mentions this as an argument for more complete and reliable investigations than are now recorded and in such detail as to be entirely convincing.

The economical importance in the use of a proper run-off factor is obvious. Much of the cost of the sewer depends on whether a factor of 40, 60, or 80% is chosen and in a large district an expenditure of several hundred thousand dollars may depend on the selection. Any coefficient based on population density, although applicable in a general way for densities up to 100 per acre, as shown in Table 5, would be impracticable for such developments as obtain in the four cases previously cited, some of which house very large day, in addition to resident, populations.

To summarize, it is believed that although many experiments have been made to determine run-off, the published results usually fail to show that various possible causes of error have been eliminated and indicate that there is need of more definite and reliable data regarding the proper run-off factors for areas of different types. Gaugings are costly and must be maintained for long periods and planned with great care. For these reasons, few municipalities undertake them, and it is submitted whether, in view of its importance and general nature, it would not be an appropriate investigation to be taken up by the Federal Government.

L. Standish Hall,* Assoc. M. Am. Soc. C. E. (by letter).—The author has presented many valuable data and formulas connected with rainfall and run-off studies, but only that part of the paper which deals with the relation between annual rainfall and run-off will be discussed.

The writer has been especially interested in the curve in Fig. 4, which shows the frequency of wet and dry years based on the combined records of rainfall at San Francisco and Sacramento, Cal. In presenting data of this nature, their value can be greatly increased if the coefficient of variation of the series is given, since this is an aid in comparing the results with similar curves that may be deduced from other data. In a recent paper, the writer presented a series of curves showing the probable variation in yearly run-off covering a wide range of values of the coefficient of variation.† In order to compare the rainfall frequency given by Mr. Grunsky with the writer's run-off frequency curves previously mentioned, the coefficient of variation (*C. V.*) of Mr. Grunsky's series has been computed and found to be 0.35. Also, in plotting data of this kind, it is better to consider the percentage of time represented, rather than the number of years in the series, which will probably vary with each record to be studied. Plotting records on the basis of the percentage of time covered, therefore, is also an aid in reducing all records to a comparable basis.

Mr. Grunsky's curve has been replotted in the manner suggested, and the writer's curves for values of *C. V.* of 0.30 and 0.40 are also shown in Fig. 12. The agreement between these curves is close, the greatest deviation being for values between 100% and 140% of the mean. It is possible that rainfall data can be represented best by a type of skew curve different from that which applies to run-off data of approximately the same coefficient of variation. However, it is unfortunate that more data were not utilized in constructing the rainfall frequency curve, since with the many long records of rainfall available in California, a series containing 500 terms or more could have been easily constructed, which would have indicated more definitely the trend of such a curve.

The accuracy of the long-time means determined from the San Francisco and Sacramento records, moreover, is open to question. These records represent a composite of records taken at different locations and with gauges under varying degrees of exposure. The greatest factor contributing to the error

* Asst. Engr., H. L. Haehl, San Francisco, Calif.

† See Fig. 13, "The Probable Variations in Yearly Run-Off as Determined from a Study of California Streams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 244.

of the mean is that in these cities (and more especially in San Francisco), the gauges have been moved from their original locations near the ground to the tops of high buildings. Without going into the details which have already been given in previous papers,* it has been shown in the case of San Francisco, by comparing the official record with that kept by a private observer, that after the Weather Bureau had been moved to the top of the eleven-story Mills Building, the official record showed materially less catch than the private record, whereas prior to that time the two records had been in close agreement. When this subject was discussed previously, various arguments were advanced, defending the official record, but, at the present time, even the Weather Bureau officials recognize that the observations taken at the tops of high buildings do not reflect accurately the conditions near the ground; and it is now the policy of this Department to move its city stations, as fast as funds are available, to some open space where the instruments can be placed at or near the ground.

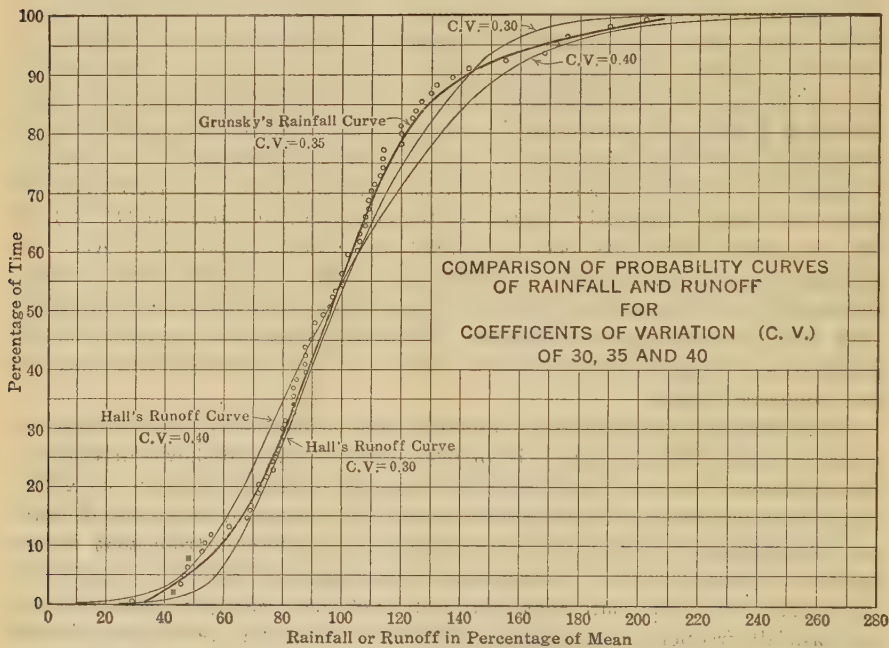

FIG. 12.

While on the subject of the effect of the exposure of rain gauges on the catch registered, attention may be called to an investigation by Messrs. William G. Reed and Howard M. Loy.† This study was originally undertaken for the purpose of making an intensive investigation of the relation

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 526-529; and Vol. LXXXI (1917), p. 183.

† "The Water Resources of Strawberry Creek, Berkeley, Cal.", *Monthly Weather Review*, Vol. 43, pp. 35–39; and "Note of Effects of Raingauge Exposure", *Monthly Weather Review*, Vol. 43, pp. 318–322.

between rainfall and run-off on a small drainage area. Strawberry Creek, a small intermittent stream which has a drainage area of about 600 acres, and which flows through the campus of the University of California at Berkeley, Cal., was selected for this purpose. The work was begun in 1913 when five gauges in addition to two permanent gauges on the campus were placed in different parts of the drainage area and read regularly. At the same time the flow of the creek was measured by a weir. The results obtained from the five rain gauges were so unsatisfactory that, in 1914, thirteen gauges were placed on the drainage area, in addition to the two on the campus. In Table 15 is shown the rainfall recorded in the various gauges between May 1st, 1914, and April 30th, 1915, together with the elevation of each gauge and the ratio of the catch of each gauge to the average. The term, expanded total rainfall, which is used, refers to the total recorded rainfall plus certain corrections to fill in gaps in the records at some of the gauges. The variation in the ratio of the catch of the various gauges for individual storms was often much greater than the average given in Table 15, and the records show that the lowest catches are in the gauges most exposed to the wind. It was stated as doubtful whether there was a single good rain-gauge exposure in the area under consideration, and it was believed that all the gauges registered less than the true rainfall. As a conclusion to this study, which was abandoned shortly afterward, Mr. Reed states:

"From a strictly meteorological point of view the most important result so far seems to be the difficulty if not the impossibility of determining the precipitation on a water-shed by means of ordinary rain gauges."

TABLE 15.—Rainfall of Strawberry Canyon from May 1st, 1914, to April 30th, 1915, with Ratios of Individual Gauges to the Average.

Gauge number.	Elevation, in feet.	Expanded total rainfall, in inches.	Ratio to average.
Students' Obser	325	26.91	1.00
Civil Engineering Building	410	24.49	0.91
1	520	28.79	1.07
2	730	28.79	1.07
3	880	26.91	1.00
4	1 225	26.10	0.97
5	1 270	22.60	0.84
5	1 270	21.53	0.80
6	1 250	27.18	1.01
7	1 190	25.30	0.94
8	1 180	28.26	1.05
9	915	29.06	1.08
10	1 210	29.06	1.08
11	1 315	32.03	1.19
12	1 655	29.06	1.08

It would appear, therefore, that great doubt exists as to the accuracy of long-time means based on composite records. Hence, it would seem that there is nothing to be gained by expanding rainfall records to agree with the long-time means at San Francisco and Sacramento.

It may be of interest to discuss in some detail the results of a study of the relation between rainfall and run-off on Putah Creek, as this stream shows some of the typical difficulties encountered in studies of this nature.

The records of the gaugings of the flow of Putah Creek at Winters, Cal., are available for a period of fifteen years from 1905-06 to 1919-20. The record for a period of nine years has been presented by the writer.* The run-off of the remaining six years need not be shown, except to state that, in 1919-20, the run-off was slightly less than in 1911-12, and that the mean run-off for the fifteen years is 420 000 acre-ft. In order to expand this record over the period covered by the rainfall records, the precipitation by seasonal years for eighteen stations in the neighborhood of the drainage area was collected. After gathering these data, it was found that only seven stations out of the eighteen had complete records for the fifteen years that run-off records were available. Gaps in the records were filled by estimating the missing years from the rainfall at neighboring stations.

Table 16 gives the names of the various rainfall stations, together with the elevation and the mean annual rainfall for the 15-year period. Only one of the eighteen stations, that at Helen Mine on the north side of Mount St. Helena, is within the catchment area, and at this station the mean rainfall is 82.3 in., or more than twice the mean rainfall at any of the other stations. Helen Mine is the only station on a mountain slope, all the other stations being in the bottoms of valleys. Without this station an erroneous idea of the true mean rainfall in the Putah Creek basin would have resulted. From the scarcity and wide distribution of the rainfall stations, it is almost impossible to estimate the mean annual rainfall on this drainage area with any degree of accuracy, although it is probable that it is between 30 and 40 in. In cases of this kind, better results can be obtained by disregarding the actual rainfall and determining the relation between the percentage of rainfall and the run-off.

TABLE 16.

Station.	Elevation, in feet.	Mean annual rainfall 15-year period, in inches.	Station.	Elevation, in feet.	Mean annual rainfall 15-year period, in inches.
Suisun	15	19.80	Peachland	190	39.59
Napa	50	23.12	St. Helena	255	35.32
Davis	51	17.24	Cloverdale	340	38.89
Healdsburg	52	38.66	Guinda	350	20.37
Woodland	63	16.24	Calistoga	363	36.98
Dunnigan	65	19.07	Ukiah	620	36.75
Sacramento	71	15.95	Upper Lake	1 350	27.16
Vacaville	175	24.73	No. Lakeport	1 450	28.12
Santa Rosa	181	28.42	Helen Mine	2 750	82.29

In Fig. 13 is shown the relation between mean annual rainfall and mean annual run-off for Putah Creek, based on the 15-year period covered by the gaugings. The relation between the annual rainfall and run-off has not been shown in the diagram, because the only use that is made of the rainfall run-off relation in this discussion is to determine the relation between the observed and the probable run-off for the period covered by the rainfall records. By using the data in the manner illustrated in Fig. 13, it has been found that

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), Stream 11, Table 1, p. 194.

in practically all cases the relation between the percentage of mean annual rainfall and the run-off can be represented by a straight line.

The percentage of rainfall for each year of the records at each station was calculated on the basis of the 15-year means, and these percentages were then averaged to obtain the average percentage of rainfall for each year. For the 48 years, 1872-73 to 1919-20, there were always at least three stations available for computing the average, but for the 23 years, from 1849-50 to 1871-72, only the record at Sacramento was available. Using the records in this manner, it appeared that for the 48-year period the rainfall was 103.5% of that in the 15-year period, and that for the 71 years it was 109.0% of that in the 15-year period.

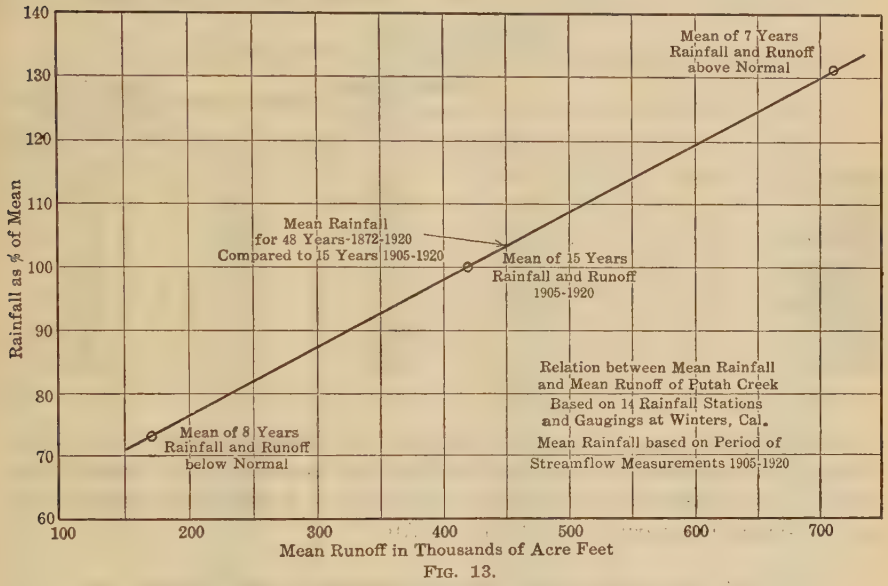

FIG. 13.

From a closer study of the records, it would appear that the conditions which have affected the catch registered by the San Francisco gauge in recent years, that is, moving the gauge to the top of a high building, have affected the Sacramento record in a similar manner. In support of this statement, Table 17 is offered, showing the ratio between the mean rainfall for the fifteen years, 1905-06 to 1919-20, and the mean for the entire record at each station where the records were of sufficient length to warrant making the comparison. The record for Sacramento is shown for two periods, 48 years and 71 years.

It will be noted from Table 17, that the ratio of the rainfall for the entire record to that of the last fifteen years is disproportionately greater in the case of Sacramento than in that of any other station, if either the 48-year or the 71-year period is considered. In view of these results, it would appear that nothing but erroneous results could be obtained by expanding short records to agree with the long-time means at San Francisco and Sacramento.

It was decided, therefore, in the case of the Putah Creek study, that the results obtained from the 48-year period were much more dependable than those for the 71-year period. Using the result for this period and Fig. 13, it would appear that the mean annual run-off for the 48 years should be 450 000 acre-ft. rather than the 420 000 acre-ft., as determined from the measured flow.

In Table 4, Mr. Grunsky has presented the probable range in rainfall and run-off to be expected, in a period of 100 years, from a water-shed in the vicinity of San Francisco having a mean annual rainfall of 30 in. Similar results may be obtained for the run-off—which is really the result finally desired—by using the writer's frequency curves for run-off.* For streams in the vicinity of San Francisco, the curve for the coefficient of variation of 0.70

TABLE 17.—Summary of Rainfall Records near Putah Creek Drainage Area.

Station.	Mean, 15 years, 1905–20, in inches.	Years in record.	Ratio of mean entire record to mean, 15 years.
Sacramento	15.95	48	115.9
Sacramento	15.95	71	117.6
Davis	17.24	48	98.8
Vacaville	24.73	40	106.7
Suisun City	19.80	46	101.9
Napa	28.12	43	103.5
Dunnigan	19.07	43	103.5
Woodland	16.24	47	108.1
Upper Lake	27.16	34	101.1
Calistoga	36.98	47	99.3
Ukiah	36.75	43	99.5
Healdsburg	38.66	43	107.4
Santa Rosa	28.42	32	106.3

is the proper one to use. In order to support this statement, statistics on some of these streams will be set forth in Table 18, and it will be noted that although some differences occur in the run-off, the coefficient of variation is very consistent and varies but little from 0.70.

TABLE 18.—Summary of Run-off Records in the Vicinity of San Francisco, Cal.

Stream.	Place.	Years in record.	Area, in square miles.	Mean run-off, in inches.	C. V.
San Leandro	Oakland	34	42	9.0	0.72
Crystal Springs	San Mateo	30	36	11.1	0.69
Alameda	Sunol	29	620	4.8	0.72
Coyote	Madrone	13	193	7.5	0.80
Putah	Winters	14	805	10.5	0.67
Cache	Yolo	16	1320	7.8	0.74

Applying this method to Putah Creek, Table 19 shows an ideal 100-year record of annual run-offs for this stream, arranged in order of magnitude, and based on a mean annual run-off of 450 000 acre-ft., and a coefficient of variation of 0.70.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 244.

The percentages given in Table 19 will apply equally well to any stream having a coefficient of variation of 0.70, and the yearly run-off can be obtained by multiplying these percentages by the mean annual flow. The method of obtaining these percentages has been explained in the writer's paper previously mentioned.* The minimum run-off to be expected once in 100 years on Putah Creek at Winters, or 13 000 acre-ft., is equivalent to a depth of 0.30 in. on the drainage area, while the maximum run-off, or 1 521 000 acre-ft., is equivalent to a depth of 35.5 in. on the drainage area. The figures given in Table 19 may be used in constructing an artificial record of any desired length, which will be comparable to an actual record of similar length obtained under natural conditions.* Such a record is interesting and instructive, as from it the probable frequency of the re-occurrence of any series of dry years can be foretold with reasonable accuracy.

TABLE 19.—Ideal 100-Year Record of the Annual Run-off of Putah Creek.

Order of magnitude.	Percentage of mean.	Run-off, in acre-feet.	Order of magnitude.	Percentage of mean.	Run-off, in acre-feet.	Order of magnitude.	Percentage of mean.	Run-off in acre-feet.
1	3	13 000	35	58	261 000	68	120	540 000
2	8	36 000	36	60	270 000	69	123	553 000
3	11	49 000	37	61	275 000	70	125	563 000
4	14	63 000	38	62	279 000	71	128	576 000
5	16	72 000	39	63	283 000	72	131	589 000
6	18	81 000	40	65	293 000	73	134	603 000
7	20	90 000	41	67	301 000	74	137	616 000
8	22	99 000	42	68	306 000	75	140	630 000
9	24	108 000	43	69	311 000	76	143	643 000
10	25	113 000	44	71	319 000	77	146	656 000
11	27	121 000	45	72	324 000	78	149	671 000
12	29	131 000	46	74	333 000	79	153	687 000
13	30	135 000	47	75	337 000	80	157	707 000
14	32	144 000	48	77	347 000	81	161	725 000
15	33	149 000	49	79	355 000	82	164	738 000
16	34	153 000	50	80	360 000	83	168	756 000
17	36	162 000	51	82	369 000	84	172	774 000
18	37	167 000	52	83	373 000	85	177	796 000
19	38	171 000	53	85	383 000	86	181	814 000
20	40	180 000	54	87	391 000	87	186	836 000
21	41	185 000	55	89	401 000	88	190	855 000
22	42	189 000	56	91	409 000	89	195	877 000
23	44	198 000	57	93	419 000	90	200	900 000
24	45	203 000	58	95	427 000	91	206	926 000
25	46	207 000	59	97	437 000	92	210	945 000
26	47	211 000	60	100	450 000	93	217	977 000
27	48	216 000	61	102	459 000	94	224	1 008 000
28	50	225 000	62	105	473 000	95	231	1 039 000
29	51	229 000	63	107	481 000	96	240	1 080 000
30	52	234 000	64	110	495 000	97	250	1 125 000
31	54	243 000	65	112	504 000	98	262	1 168 000
32	55	247 000	66	115	517 000	99	285	1 283 000
33	56	252 000	67	118	531 000	100	338	1 521 000
34	57	257 000	.....	.....	.....	.....	.....	.....
Mean.....							100	450 000

Some space has been devoted to a discussion of the desirability of having the Weather Bureau publish rainfall totals by climatic or seasonal years rather than by calendar years. Although the annual summaries published by the Weather Bureau give the precipitation by calendar years only, bulletins of climatological data are issued at intervals of a few years, in which the entire

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 250–251.

record of rainfall at each station to the date of publication is given both by seasonal and by calendar years. In order to bring the records to date, it is generally only necessary to secure the seasonal totals over a short period of years.

H. V. Hinckley,* M. Am. Soc. C. E. (by letter).—The Engineering Profession is certainly indebted to the author for making public the results of his investigations. The writer would not attempt to estimate the loss of capital invested in hydraulic works, during the past thirty or forty years, from lack of such information.

Time was when, in the absence of more complete information, the evaporation tests which had been made by water supply engineers along the Eastern seaboard—and which were widely published—were accepted as probably applicable over the United States in general. This mistake, coupled with a lack of appreciation of the distinctive differences between rainfall and run-off in different sections of the country, has been partly responsible for great losses.

The conditions prevailing in New England and the seaboard States, are quite dissimilar from those which prevail on the Great Plains. Even the casual observer who travels, knows that in the Eastern States the streams flow the year around, while, in parts of the West, the streams are dry fifty-one weeks of the year and, human-like, make up for lost time by getting soaked and going beyond customary limits during a part of the remaining week.

The writer has previously referred† to the comparative fluctuations of the Connecticut and Kansas Rivers. On the Great Plains, extremes prevail. Drainage areas, reservoir capacities, and spillway capacities must be maximum, while at times results as to water supply will be minimum. For example, since 1900 there have been two years when the run-off in parts of Oklahoma, for sixteen months (including four months of the contiguous years, in each case), has been less than 4 in., while the evaporation for the same period has been about 10 ft. The writer discovered, in the early Nineties, that an average annual evaporation of 3 ft., or a maximum evaporation of 6 in. per month, did not apply to the case of hot winds on the Plains where evaporation was as high as 2 in. per day, for 15 or 20 days at a time.

The author's formula for annual run-off, in which P is less than 50 in., has always checked the writer's computations (based on actual measurements at neighboring points similarly situated), to within a small fraction of 1%, and those cases include the two extremely dry years previously mentioned. The writer, therefore, considers Mr. Grunsky's simple and easily remembered run-off formula one of the best tools in his engineering collection, and believes that it entitles the author to at least a passing compliment.

Walter E. Jessup,‡ Assoc. M. Am. Soc. C. E. (by letter).—The author has collected and evolved formulas expressing relations between rainfall and run-off, which can be made to be very useful. It is desired, however, in connection with the examples presented by Mr. Grunsky of applications of

* With V. V. Long & Co., Oklahoma, Okla.

† *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 34.

‡ Asst. Engr., Quinton, Code and Hill, Los Angeles, Calif.

his Equation (25), namely, $D_m = 413\ a\ M\ I$ sec-ft., giving maximum stream flow from the intensity of a rainfall on a large drainage area, to point out that this formula has its limitations.

The author has given, as examples, the drainage areas of the American River above Folsom and of the Sacramento River above Red Bluff. As stated, the greater part of the yearly precipitation on the Pacific Slope occurs during the winter months. The average elevation of the 1 900 sq. miles of the water-shed of the American River, is nearly 5 000 ft. above sea level. Therefore, a large part of the precipitation will fall as snow and will not run off at once. The average elevation of the 9 300 sq. miles of the water-shed of the Sacramento River is about 4 000 ft. above sea level, and a large part of the precipitation takes the form of snow for similar reasons. The floods on both these rivers occur for the most part in the spring of the year as a result of melting snows.

Assuming that the maximum rainfall of 8 in. in 24 hours should fall in the form of rain, on the water-shed of the American River in the spring, there will be added to the run-off from such a rain, the tremendous run-off from the melting of the accumulated winter snows, the result being a value far beyond the expectation of the formula. On the other hand, should this same precipitation occur during the winter, it will be retained largely on the water-shed in the form of snow, and only a small proportion of the run-off, as computed from the formula, will result. Such unreasonable results will also obtain from a maximum rainfall of 6 in. in 2 days on the water-shed of the Sacramento River.

It would seem, therefore, that the application of the formula has been carried beyond a reasonable limit, and that it will not apply to such large water-sheds where the conditions affecting precipitation are so variable.

E. W. Clarke,* M. Am. Soc. C. E. (by letter).—The writer is interested in Mr. Grunsky's suggested method of expanding short-period rainfall records, and although it is understood that special reference is made to California conditions, it may be worth while to compare the theoretical results obtained by the expansion with the rainfall at some long-term stations in New England, using still longer records at other stations as a base. It is evident that this method of expanding rainfall records may also be applied to run-off records.

X. H. Goodnough, M. Am. Soc. C. E., Chief Engineer of the Massachusetts State Board of Health, has collected a large number of rainfall records in New England,† some of which are as follows:

New Bedford, Mass.	1814–1920
Boston, Mass.	1818–1920
Providence, R. I.	1832–1920
Lowell, Mass.	1826–1920
Amherst, Mass.	1836–1920
Waltham, Mass.	1825–1920
Springfield, Mass.	1846–1920
Concord, N. H.	1853–1920

* Dist. Plant Engr., U. S. Shipping Board, Baltimore, Md.

† *Journal*, New England Water Works Assoc., September, 1915, and September, 1921.

Using as the base time, the 50 years, 1871 to 1920, inclusive, and as the base stations, New Bedford and Boston, the rainfall was calculated to the limit of each of the other stations from one or the other of the base stations and sometimes from both; also, in a few cases, by using one of the sub-stations as a base, and the results of these calculations and the actual recorded rainfall were plotted.

The plotting of the annual rainfall showed that, in general, a wet or dry year at the base was also a wet or dry year at the sub-station, but the theoretical and actual quantities are often far from agreement. In cases where the theoretical rainfall was calculated from both bases, there is also generally a lack of agreement between these figures.

The mass curves showed that there may be practical agreement for a number of years, but this agreement never follows throughout the entire period and it happens, in most cases, that the difference is on the wrong side, that is, less rain fell than the calculations indicated as probable.

The maximum distance between stations is about 95 miles. At the various stations, rainfall conditions differ, but the writer does not consider them so dissimilar as to make it unfair to the theory to include them in the test. If the application of the theory is confined to much smaller areas than this, its value is nullified. So far as these few cases indicate, distance within these limits does not seem to be a factor, since the variation between the theoretical and actual at Amherst, calculated from Boston 75 miles away, is no greater than at Waltham, 12 miles distant from Boston.

This test of the theory apparently indicates that, as far as New England records are concerned, reliance cannot be placed on this method of expanding short-term records to give even approximate estimates of the precipitation for the years not covered by the record. In view of the general discordance of the theoretical and actual rainfall lines, not much reliance can be placed on the calculated maxima and minima; on the contrary, at one station the maximum and at four stations the minimum estimated, was exceeded by the actual rainfall.

W. W. Horner,* M. Am. Soc. C. E. (by letter).—The writer feels that Mr. Grunsky's paper, as supplementing his earlier paper on the same subject,† is of the greatest interest to the Engineering Profession, particularly in keeping before engineers the lack of information available on certain phases of this subject. With the adoption of the rational method of determining run-off, a great step was made in advance in this particular science. For its satisfactory application there are two fields in which study is still required, the first relating to intensities and duration of rain, and the second to the rates of run-off.

With the increasing number of automatic rainfall gauges installed, knowledge of the variations of rainfall intensities, with respect to the duration of the rain, is rapidly approaching a satisfactory condition. The writer has made a study of a number of records for various localities, and although Mr. Grunsky's parabolic formula of variation does not appear to conform closely

* Chf. Engr., Sewers and Paving, St. Louis, Mo.

† "Rain and Run-Off near San Francisco, California", *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 496.

to general data, the writer believes that its use will be valuable in a preliminary way in districts where detailed studies are not available. He agrees with the author that it should never be used where sufficient observed data are available to permit of a satisfactory rainfall curve for the particular locality, or where such curves are available in the same general district and under the same meteorological conditions.

Another phase of the rainfall studies on which little information is available is the intensity and area of relationship, that is, a determination of the area over which single-station maximum records are applicable, or the extent to which they should be reduced in applying them to large areas. In a number of cities, a sufficient number of automatic rainfall gauges are now located to permit of a study of this kind. It is being undertaken in St. Louis, Mo., in the analysis of ten-year records from five gauges, and it is hoped that, in another year, something of value may be available from this source.

In the matter of percentages of run-off, or of rates of run-off for particular intensities, the writer feels that little progress has been made. Engineers are still dealing almost entirely with arbitrary values, or, in a few instances, with values based on personal experiences. Published gaugings on existing sewer districts are few, and many of the records are faulty. Even where the records are unquestionable, the results for different storms vary so widely that engineers have not yet been able to make them conform to a definite theory. The writer has no doubt that as such records multiply, accompanied with all the information which may be of interest as affecting run-off, some definite values will be developed.

The studies of the first three years of gaugings in St. Louis were rather discouraging, and the matter was laid aside for the accumulation of further data. With records for ten years now available, it is again hoped that some law of variation in run-off will become discernible. These gaugings, like most of those heretofore recorded, deal with the run-off of mixed urban areas of from 20 to 500 acres in extent. The writer has long felt that the analysis of these records would be much simplified by a study of the run-off of small plots of uniform character. In an effort to secure such data, the run-off from single blocks is being studied in St. Louis, and it is also proposed, in the near future, to make a study of the run-off on large roofs. To complete this series, observations will be made of the run-off from small plots of natural soil.

The writer has been interested in the results of the study of this character made by the Miami Conservancy District and published in Part 8 of its Technical Reports. These results give definite information of the effect of varying intensities of rainfall on run-off from natural soils in the vicinity of Dayton, Ohio. It is surprising to note the capacity of these soils for continued absorption of surface water and for passing this water through to the underlying gravels. The measurements show clearly that, for the shallow topsoils and glacial gravels of this part of the United States, the percentage of run-off will decrease with the decrease in intensity, or, in general, with the duration of the storm. It should be evident, however, that this variation is

not applicable outside these particular soil conditions, and it is to be expected that, with the surface loams and plastic clays found generally south of the Ohio and Missouri Rivers, saturation may be expected in a comparatively short time, and that, in most instances, the percentage of run-off will increase with the duration of the rain.

The writer feels that this latter relation will also apply to most urban areas, and that some variable factor of run-off must be introduced into the application of the rational method, rather than values such as the *a* suggested by Mr. Grunsky, which are constants for any particular density of population.

There is no doubt that engineers must take account in some way of the reduction of run-off due to water in storage on the surface, and to a minor extent in the conduits. Mr. Grunsky's factor of 0.645 is apparently a secondary run-off coefficient to allow for this effect. Ultimately, the propriety of any constant value for this purpose may be questionable. With the present knowledge of the subject, however, Mr. Grunsky must be commended for his effort to take this into account.

The writer feels that Table 6 is unfortunate, in that it shows values extending far beyond the range which the author claims for his formula. It would seem preferable to have introduced only the lower part of this table, dealing with times of from 1 to 20 days. The writer feels the more strongly on this point because he does not believe that the marked reduction of the values of *a* with an increase in time are justified for rains of less than 1 day, and seriously questions whether, for these short times, the values of *a* should not increase somewhat as the time increases.

The writer's experience with the application of the rational method in the design of sewers in St. Louis has led him to the conclusion that the simple and direct use of the method involves less work than the solution of formulas of the type presented by Mr. Grunsky. It would seem unfortunate to have to make preliminary studies of the water-shed for the purpose of determining the time of flow, and of drainage areas which would correspond to fractions of this time, when in the ordinary use of the rational method the solution in each section of the conduit provides all the data of this type required for the succeeding section. It appears that the use of Mr. Grunsky's formula will involve about twice the arithmetical work required by the simple application of the rational method.

C. E. Grunsky,* M. Am. Soc. C. E. (by letter).—The statements of Mr. Hinckley with regard to evaporation in parts of Oklahoma, should be taken with some allowance. Even in the hot, dry, almost rainless, and quite windy Salton Basin, California, the mean monthly temperatures of which are higher than those in Oklahoma, the annual evaporation from an open water surface is only about 6 ft. The writer would regard, as highly improbable, the evaporation of 10 ft., from an open water surface of any considerable depth and extent in 16 months, including 4 months of the contiguous years. This is also true of a rate of 2 in. per day for 15 to 20 days at a time. Such

* Cons. Engr., San Francisco, Cal.

rates would obtain only from small experimental apparatus or from soil kept wet almost to the saturation point.

Possibly Mr. Jessup is correct when he assumes that the greatest discharge of such a stream as the American River, California, will occur when a heavy rain falls on snow. The maximum melting effect will be due to a so-called warm rain; but the rain of maximum intensity is usually the cold rain which comes near the end of the storm. It will probably be found to be true that rain water plus water due to melting snow will never exceed the predicted maximum rain intensity for the entire water-shed. The checking of the suggested formula against any known results, where melting snow contributes to the peak discharge, is invited. If maximum rain intensities for the water-shed are properly determined and if topographic features have been properly considered in estimating the critical time, it is believed that the formula will be found to be dependable, even under the condition suggested by Mr. Jessup.

The statement by Mr. Clarke that the method of expanding short-time records to long-time records is applicable in the case of run-off as well as of rainfall, is true, but the data relating to measured run-off are accumulating slowly and in many parts of the United States the relative quantity of annual water production can only be determined for the long-time period from the rainfall records, which are generally available. He is correct when he states that the computed quantities of rain for any sub-station will frequently depart materially from the measured quantity. This cannot be otherwise; but it does not detract from the value of the computed quantity (when no measured quantity is available) as an element for predicting what may be expected in the future. Furthermore, as stated by the writer, there is less chance of agreement between any computed rainfall and the measured rainfall for a station, in regions such as the Atlantic Coast and the Middle West where thunder-storms, usually of small extent, occur. This, however, does not destroy the serviceability of the method as a basis for predictions of what may happen in the future. It must always be borne in mind that the purpose of studying rainfall and run-off of past years is to lay a foundation for the prediction of future probabilities. It can matter little, therefore, if predictions for individual seasons depart considerably from measurements. It is the possible sequence of seasons of various quantities of rain and run-off that is of consequence.

The writer is in accord with Mr. Wood as to the value of the study of flow records where they are available for a sufficiently long series of years. They are particularly valuable for the streams in the Eastern United States, the range of discharge of which is much less than that of streams of the Pacific Coast. However, it will hardly do, as he suggests, to use the run-off records of streams from other regions as a basis for expanding a short-time Pacific Slope record. The first requisite, in such comparisons, must always be similarity of climatic and general topographic and geologic conditions.

The reasons given by Mr. Wood for adhering to the calendar year, in the matter of rainfall and run-off records, have little to commend them. The calendar year is good enough perhaps for the Southern Hemisphere where

the New Year comes in a low-water period. It was a mistake to have adopted it for the Northern Hemisphere, and the mistake should not be perpetuated. The improved practice, for the reasons stated by the writer, should be adopted, but the division of the year that is intended and that should be adopted, should be the same for all parts of the United States.

The writer is particularly gratified to note that the U. S. Weather Bureau is giving careful consideration to the matter of revising the method of tabulating observations, as shown by the comments of Dr. Marvin, Chief of the Bureau, and some assurance has been given by the Bureau that in California the regular publications, if not modified, will at least be supplemented with a special publication of the rainfall data in more useful form.

The discussions of Messrs. Hering and Landreth are of interest, but the writer does not wish to be understood as claiming that evaporation is the only factor that causes less water to run off than has fallen, as rain or snow, on any area. What is claimed is that evaporation is the predominating factor and that transpiration, the assimilation of water by growing plants, and evaporation, may be grouped together and for convenience called "evaporation". The causes making for reduced evaporation at high altitudes cited by Professor Landreth are offset in some measure by the fact that, at the high altitudes, snow remains long on the ground and there may be consequent replenishment of soil moisture long after the rain has ceased. It is not believed that any attempt to introduce other factors than temperature, which is the main index of the rate of evaporation, into the equation will meet with success and if the coefficient appearing in Equations (13) and (14) has been determined for one or more water-sheds of any region, these equations should be found useful as written.

Mr. Horner's suggestion that the value of the permeability factor, a, should not be constant for any density of population, will readily be conceded. The factor to use and how to vary it, must be left to the judgment of the engineer and the degree of approximation that is desired. The variability of this factor is recognized by the writer, as shown in Equation (29), and the resulting Table 6. Mr. Horner suggests, too, that Table 6 has been extended too far. Experience alone will determine this on the basis of comparison of measured maximum discharge with the estimated discharge, when the facts relating to rainfall on the water-shed are sufficiently well known to make a comparison.

As an addition to the paper, it should be noted that, in using Equation (13) for run-off estimates, the mean temperature of no month should be entered at less than 32° in the calculation of the value of f. It should also have been stated that the coefficient, C, in Equation (14), is probably inversely proportional to n, the number of months, and that, consequently, Equation (13) is applicable regardless of whether the rain that produces the run-off, falls in 6 or 7 months, as on the Pacific Coast, or is more uniformly distributed to all the calendar months, as in other parts of the United States.

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NATIONAL PORT PROBLEMS*

A SYMPOSIUM

By Messrs. Frederick W. Cowie, Lansing H. Beach, Frederic H. Fay, M. A. Long, J. Rowland Bibbins, John Meigs, W. Watters Pagon, Edwin J. Clapp, Carroll R. Thompson, John A. Bensel, William J. Wilgus, B. F. Cresson, Jr., and H. McL. Harding.

With Discussion by Messrs. William H. Adams, Arthur M. Shaw, T. F Keller, Harwood Frost, L. F. Bellinger, John H. McCallum, Frank W. Hodgdon, M. G. Barnes, Nelson P. Lewis, and T. Howard Barnes.

* Presented at the meetings of September 7th and 8th, 1921.

NATIONAL PHASES OF PORT PROBLEMS

By Frederick W. Cowie,* M. Am. Soc. C. E.

What is a National port? National ports stand for policy, also projects.

In recent years, increasing productiveness, the tremendous increase in foreign commerce, and the increase in population, wealth, and industrial activity, has not been followed, or even equalled, by the commensurate, scientific development of ports.

The transportation problem of a nation is one of the fundamentally vital economic questions of the day. It is not one of limitations as regards the individual or the locality. It gives additional prices to the farmer for his produce; it enables the manufacturer to compete with his foreign rivals; it cheapens the necessities of life; and it gives employment to labor and capital alike. A successful National port, therefore, is widespread in its effect for good. It may and frequently does constitute the deciding factor as regards a competitively successful National route.

A National port is an important link in the transportation system of a nation. A successful National port is a distinct aid in the solution of the transportation problems of a country.

A National port must bear a clear relation to a logical, economical, and widespread necessity. To be truly National, a port, or series of ports, must be required by the nation and directly or indirectly it must yield comprehensive advantages. In consequence, few port locations fulfill the necessary requirements. A port, however, may be National in part for comprehensive collective and distributing functions, and local in part to fulfill city or State requirements.

Although a National port is of general value to the nation, it is obviously of particular value to the city in which it is located and to the hinterland which it serves. The district through which the transportation route runs collects transportation tolls all along the line. To distribute the benefits and to divide the burdens alike has ever been the difficulty.

A State, a city, or even a corporation, may decide, in view of expected financial and commercial results, to develop an advantageous natural harbor, even though no great comprehensive or National economic interests would be served. In no sense could this be considered a National port. On the other hand, it would be logical to infer that a comprehensive unit of a port may be developed to meet National requirements and that, in the same harbor, other units may be developed to meet local or more city requirements. A combination port developed and administered, alike for National and for restricted requirements, both as to benefits and the assumption of financial burdens, would give ideal results. The one may not be successful without the other. The combination results in excellent working conditions.

An island nation having only short distances to the seacoast may be better served by a number of local ports. Transportation is not a problem in this case. No comprehensive general interests would be served by the development of one or more particular ports. All producers and consumers have the

* Chf. Engr., Harbor Commrs. of Montreal, Montreal, Que., Canada.

competitive advantages of short transportation distances and cheap freights. On the contrary, a country having an extensive hinterland and a short seacoast and having rival foreign competitive transportation routes, essentially requires a system of National ports. The transportation problem in this case is to find the best National route, develop it, cheapen it, and use it. Transportation tolls are kept in the country and constitute a National advantage, even though the foreign route may be cheaper to some extent. Many rich hinterland areas cannot be developed successfully or opened up for settlement and production, without National aid toward cheap transportation. In particular, therefore, a country having immense transportation distances and productive hinterland areas, vitally requires a National transportation policy, with more or less clearly defined National routes, wisely developed National ports, scientifically improved, equipped, and operated in the interests of a nation. Such examples as the United States, Canada, Brazil, Russia, and Germany may well be cited.

In the United States the genius of commerce is one of the important characteristics of the times. It is a function of the nation rather than of the individual. National prosperity is vitally dependent on commerce. Transportation is one of the important phases of commerce. A transportation policy is essential. This policy, as to its National well-being, may be compared with its fiscal or banking policy. Production may be hampered; manufacturers may not be able to compete; the necessities of life may cost too much, if a satisfactory solution of the transportation problem is not achieved.

The so-called "pork barrel policy" is directly and indirectly the antithesis of the National port policy. If certain ports, natural or artificial, can, by scientific development, be made by the nation of value to the people in general, such development will be in line with a National port policy. It does not follow, however, that because of National expenditure on these certain ports, similar or proportionate National expenditure must be made on others which are not so essential in connection with the adopted National routes.

National ports must be connected with trunk lines, not with local lines or feeders. Is it reasonable to expect that a nation can be asked to sanction such a National port policy which, though general in its good results, must be more generous to some ports than to others? History demonstrates an affirmative reply. A necessary project for the country will command universal approval, even at some sacrifice of the few. Admiral William S. Benson, U. S. N., is recorded as saying, after seeing other harbors and ports of the world: "We [the United States] have not really a first-class harbor, *viz.*, scientifically constructed and developed."

The speaker has seen and studied most of the great harbors of the world and, in his opinion, no country, other than the United States, except possibly Canada, has greater necessity for scientifically developed harbors and, in no country of the world, can be found more favorable opportunities. Transportation is fundamentally essential; distances are great; and routes are well defined. A single great city in the United States possesses two of the most magnificent railway passenger terminals in the world. They are so splendid, so successful, so representative of the spirit of the people, that it is

incomprehensible that there is not one or more similarly representative ocean terminals. It may not be a nation's business to build something splendid for one city, or one State, or one steamship line, or one railway system, but a splendid ocean terminal in the greatest port in the world, at the gateway to three-quarters of the passenger commerce of the country, is a sane vision. Moreover, if properly co-ordinated, with a comprehensive National port unit, it would be an extraordinarily attractive and paying proposition.

Canada, with her great central but rich productive areas, her necessarily long east-and-west transportation, and her limited financial resources, is making wonderfully successful progress. There are notable examples of National ports in Canada. There are others partly National and partly local and they are making wonderful strides in helping to solve that most critical Canadian effort, *viz.*, the development of her vast areas in order to enable her people to compete in production and industry with other nations of the world.

Brazil is in the process of development and is successfully exploiting the nebulæ of what in time will become an organization which will result in the successful development of a great country.

The case of Russia is at present desperate. When the regeneration comes, if that vast country is to be successfully developed, a series of National ports on the east, south, and west will be one of her first necessities.

The Port of Hamburg, with its auxiliary, Cuxhaven, is reported to have cost the State of Hamburg little short of $100 000 000, and that the dues collected did not nearly pay the expenses of the port system. It was, however, a National port, and the deficit was cheerfully met from other sources of revenue. A similar courageous and wise policy of port development in America would surely result in widespread and remunerative trade and commerce, both in the United States and in Canada. There is no example in ancient or modern times of such a proportionate trade and commerce as was developed in Germany from 1888 to 1914. A settled purpose made itself manifest; a National transportation policy was adopted; routes were developed and encouraged for the purpose of increasing trade and for conducting it within the limits of the Empire. National privileges were granted to certain ports making them National ports, and the Empire prospered.

A National port, therefore, may be defined as one of the most important factors in the solution of the National transportation problem. This problem does not exist in certain countries. It is of vital economic importance for the prosperity and development of other countries having large productive areas and long transportation distances. What question would require more serious study, greater skill, or be more in the general public interest? The engineer has great responsibilities; he should be well posted when called on for advice.

Accommodation and Facilities Required by a National Port.—In order to measure justly what facilities may be required to take care of National port business, the characteristics of that business may be enumerated:

1. Ocean passenger business.
2. Mails and express.
3. Exports.
4. Imports.

5. The storage and handling of grain.
6. The storage and handling of perishable products.
7. General warehousing.
8. Adjuncts necessary for an ocean port.
9. Facilities and accommodation for collecting, receiving, assembling, re-manufacturing, conditioning, and distributing both from and to all ports of the world and from all points in the country.

Reduced to a single picture, the requirements for a National port may be visualized as follows: One or more comprehensive units capable of taking care of all the varied port business, co-ordinated and concentrated in a central location on one of the main natural routes, where the ship and the railway meet and where commerce may be collected, stored, and handled with economy and dispatch. It would appear, therefore, if this argument is accepted, that much of the port development of the present day is being carried out on lines absolutely contrary to the best requirements of a National transportation policy. Take location: Are the best, most centrally located sites for port development being secured no matter what the cost? Are comprehensive units being designed, so that everything required in connection with the prompt and economical loading and unloading of ships may result? Is concentration being adopted as a necessary principle? What about co-ordination? Who is responsible for the railroads designing ocean terminals?

In North America the super-skill of the designers of everything pertaining to railroads, bridges, the development and use of electricity, the motor industry, bulk-cargo handling, sky-scraper buildings, and tunnels is an acknowledged fact. Is this super-skill in evidence in the ports?

New York City which possesses the greatest port in the world and where the lead is taken in almost every phase of originality and skill in design, is, in the opinion of many, setting a very bad example in the development of her port from the standpoint of the inland producer and consumer. The local trade of the great city is such a valuable freight and such a high proportion of the port business, that ships and shipping declare for New York, in spite of the records of economy and dispatch. Competition has not been a factor and has resulted in indifference. Necessity did not require to mother invention.

The port problem in New York, as compared with other successful National ports, such as Hamburg and even Montreal, also appears to be so simple. What is the area of the Port of New York? How many miles of water-front has it? It is reported that in the combined Port of New York and New Jersey, the harbor shore line is about 700 miles in length. The developed water-front is 380 miles long.

The Port of Hamburg covers an area, 4 miles long by 2 miles wide. The harbor of Montreal has a shore-line water-front of 34 miles, while the developed water-front is only about 12 miles in length. In Montreal, centralization is one of the essential features of success. The value of port locations or berths may be judged from wharfage revenues on goods. A pier, dock, or transit shed, and dredging, costs no more in a central location than in an outlying district. The land or shore areas required are tremendously more

costly, but modern facilities may be designed for such sites, which, by concentrating effort, may multiply usage, with good financial results.

In the twenty fully equipped berths in the harbor of Montreal, centrally located along a mile of shore front, more than 75% of the business of the harbor is carried on. The revenue from wharfages on goods alone amounts to between $40 and $50 per lin. ft. of dock front per annum, including ends, bulkheads, etc. A mile distant, where railway and city approaches are not so favorable, but where docks, dredging, wharf spaces, roadways, etc., cost just as much, the wharfages amount only to approximately $10 per lin. ft. per annum. Three miles distant, with excellent railway and city approaches, with distance as the only drawback, the revenue return is about $12 per lin. ft. per annum.

To "centralize" a summing up: The Port of Montreal records a commerce in seven months, of between one-fifth and one-sixth of the commerce of the Port of New York in twelve months, and this without congestion and with a water-front development of one-thirtieth of that of New York. In New York, as in several of the other ports doing National business, there is ample opportunity, there is necessity of centralization, concentration, and comprehensive facilities for economy and dispatch for the Nation's business.

Whose Function is the Development, Construction, and Operation of a National Port?—Assuming that a nation adopts a National transportation policy, including the system of National ports, it is logical that the nation, through its representatives, has the deciding voice in all matters pertaining to policy, finance, and control. There are various principles which may be adopted in the carrying out of this policy. The matter may be placed in the hands of a Government department; it may be placed in the hands of the engineers of the Army; or it may be taken care of by direct legislation.

In order that the policy may be absolutely impartial and in the interests only of the country in general, the consideration of this transportation policy should be placed first, last, and all the time, in the hands of men who have no special interests to serve, and who have no political aspirations. Experiences during the World War are fresh in the memory. Organization after organization under the stress of vital requirements was made subservient to new methods, dependent on direction by men who had been unusually successful in some line of business or commercial activity. This resulted in an unexampled success. Problems previously considered open for individual effort, for gain, were relegated to National conferences of mind and effort. The "dollar-a-year men" were available for the critical requirements of the war. They are ever available for honorable positions at the country's call. Theirs would be the hands into which the transportation problem could safely be confided.

Although men who have already made their mark commercially or financially may be found, who will assume an honorable position and "carry on" for the honor of their country, there is another equally important phase in the success of a National port system. Port administration and port engineering are tremendous factors affecting the success or failure of such plans as would enter into the complete National system. It is probable that in no other

branch of engineering can so many costly mistakes be found. It is probable that in no other branch of engineering have the principles of success and failure in the past been so overlooked. It is a fact that no engineering study, technical or practical, can be found so absorbing, so interesting, and so valuable, when properly developed principles are carefully adhered to. In Europe, where competition in ports is so keen and where ports in general are carried on as public obligations, port engineering ranks among the very highest in the Profession.

Furthermore, the questions of continuity and permanence in connection with the conception and scope of port projects cannot be successfully answered by a system where administrators, engineers, and other technical and practical officers are changed at frequent intervals. The European system by which the best men are obtained, and permanently retained, by which, competitively, they are called on to make good and when successful are free to act independently and according to their best judgment, would appear to be a necessary principle in National port organization.

It would appear, therefore, that the development, construction, and operation of a National port system should be considered first in connection with a National transportation policy. This policy must necessarily be under the direction of the representatives of the people. Impartial direction must be given as to its magnitude, scope, and carrying out. This organization, to be truly National, should be undertaken as a duty, with public honor as the reward. Great public corporations are so directed by men who have successfully developed the various phases of success in commercial life. Successful men, in developing and building up the country's business, may be found to formulate and recommend for legislative approval a transportation policy and to direct its administration.

As a successful port owes much of its success to an advantageous location, it follows that the authorities of that location, therefore, have a vested voice in its development and operation. The hinterland served by the port has at least an equal voice, as providing the commerce for its revenues and a share in its cost, and because of its interest in economy and dispatch. Subject to the administration of the National Board, the various units of a National port system would have full local control and carry out the National units in co-ordination with the local or city port facilities.

From the point of view of the speaker, therefore, after a life-long study of the design and operation of ports, it would appear that a country desiring the adoption of a National port system, should place the development of the policy in the hands of successful men, who for the honor of their country are willing to serve, and their policy and recommendations should be authorized according to an approved programme.

Each National port scheme, adopted by the National commission, should be placed under a local commission having one representative from the hinterland, one from the line of route, and a chairman from the port city. This local commission, subject to the programme of development, expenditure, and administration, should be free and independent and should be remunerated according to its duties and responsibilities. Cheap money would be avail-

able. As far as possible, each unit should operate at cost, but the nation which would reap the benefits, should be responsible for such interest charges as are not met. It is a recognized principle that in a plan, which is essential to the well-being of the nation, and which cannot be financially developed by a city or a corporation, must be developed, either directly by the nation, or indirectly by some aid which will enable it to stand on its own feet.

Cheaper and more prompt transportation is required by the people in America. If corporations, cities, or the State, do not provide the remedy, it will not be long before the farmers, producers, and industrial interests of the interior take a hand. The coast cities have had their turn. If the country is not satisfied, the National port policy is open for trial.

It might reasonably be suggested that it would be in order to outline that which would constitute a central concentrated, co-ordinated and comprehensive unit, which would be for the well-being of a nation.

In the first place, the location must be impartially chosen, and it must be on a National route. It is most desirable that the unit should be in the very center of a harbor, close to business and trade originations. Approaches from ocean routes, connections with all railway lines, and access from city traffic centers should be perfect, or possible of being made so.

To be self-contained, the unit should comprehend: A bulkhead passenger landing quay, having berths for at least two of the largest ocean vessels on the route, approachable at all stages of the tide, and connected with a railway passenger-and-mail special siding.

The freight accommodation should be sufficient for at least twenty-five first-class berths, all closely co-ordinated. Each berth should have transit sheds, with accommodations and facilities for entering, unloading, loading, and clearing in five days.

Railway accommodation, and vehicular and lighterage facilities should be ample and sufficient for the aforesaid traffic. Warehouses should be provided for all staple products, such as for the collection, storage, and handling of grain, flour, cotton, sugar, tobacco, etc., all connected for mechanical handling to and from the ships, and with railway cars.

Cold-storage facilities should also be provided so as to develop production in the country, and to cheapen the essentials for the population.

The high priced central sites could be made revenue producing by constructing upward, for offices, manufacturing lofts, sales rooms, exhibitions, etc. The whole unit should be of a permanent character and absolutely fireproof, in order to command low insurance rates. The style and types should be worthy of the nation. If this is what a country wants, it will have it.

TERMINALS

By Lansing H. Beach,* M. Am. Soc. C. E.

The speaker will confine his remarks largely to the duties respecting terminals as imposed by law on the War Department and on the Corps of Engineers, and in explaining that attitude toward the port and terminal situation in general.

A seaport terminal may be considered to consist of land approaches, whether by rail or highway, of the terminal structures proper, including the wharf, pier, or quay, the transit shed and the necessary supporting warehouses, and the water approach. Speaking for the moment only of the latter, it is the policy of the United States Government to limit itself to work on the outer or channel side of the main harbor line, leaving to the owner of the terminal in question the making of any needed connection between the main channel and the face of the wharf or quay.

As a rule, the Government creates the main channels, usually in the past without any assistance from the locality concerned, and once these channels have been completed, maintains them, and improves and adds to them from time to time as the demonstrated needs of traffic may demand.

The admirable and comprehensive report of the New York-New Jersey Port and Harbor Development Commission is no doubt familiar to engineers, and its plans and general recommendations are known. Having in mind what is proposed in the way of future developments in the Metropolitan District, it may be interesting to summarize briefly the principal channels already created and maintained by the United States. These include the Ambrose Channel, which is 40 ft. deep and 2 000 ft. wide; the Gedney, or Main Ship Channel, which is 30 ft. deep and 1 000 ft. wide; the Coney Island Channel, 20 ft. deep and 600 ft. wide; the Bay Ridge, Red Hook, and Navy Yard Channels, on the Brooklyn water-front, 40 ft. deep and from 1 000 to 1 200 ft. wide, this channel being now in the process of extension through Hell Gate with the same authorized depth, although, at present, excavation at Hell Gate is being limited to a 35-ft. depth by 500-ft. width. The Shell Reef Channel on the New York side of the East River, 25 ft. deep and of varying width, is intended to afford useful access to that portion of the Manhattan water-front so that it may soon be improved with appropriate terminals. The Hudson River Channel, on the west side, is 30 ft. deep at Jersey City, 40 ft. deep at Hoboken, and 26 ft. deep at Weehawken. Obviously, the projects for these later improvements should eventually be unified, but, on the whole, it may be said that the channels so far described afford ample facilities for approaching the various portions of the water-front to which they relate.

It will be remembered that in the report of the New York-New Jersey Commission recommendation is made for the development of Jamaica Bay, largely for industrial purposes. The project for Jamaica Bay, adopted many years ago, is a co-operative one, under which the Government agreed eventually to provide an entrance channel, 30 ft. deep and 1 500

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ft. wide, protected by one jetty, or, if necessary, by two jetties, and an inner channel, 30 ft. deep and 1 000 ft. wide, is to be provided by the City of New York, the United States to pay such portion of the cost of dredging the interior channel not in excess of 8 cents per cu. yd. The City further undertook to construct certain bulkheads, make certain slips, and create such fills from the available spoil as might be necessary to reclaim the lowlands bordering the Bay and to make them available for terminal development under a plan that had been proposed on its behalf by a technical commission retained for that purpose. At present, the work that has been done on this project is limited to an entrance channel 18 ft. deep and 500 ft. wide, extending inland 8 800 ft. to Mill Basin and, as yet, the City has not been able to fulfill the terms of the original co-operative agreement. Recently, however, the representatives of the City have been in conference with the agents of the United States with a view to securing early and effective action toward the completion of a considerable portion of the main 30-foot channel previously described. The matter is more fully discussed in Document No. 4 of the Committee on Rivers and Harbors, House of Representatives, 66th Congress, Second Session, in which it is pointed out that further action in regard to Jamaica Bay should be deferred until its development as an element in the progressive improvement of the entire Port of New York has been considered and decided, the underlying principle being that any comprehensive plan for the further development of the Port of New York would certainly indicate the relative importance of its various elements, and that Jamaica Bay should progress in accordance with the orderly programme thus laid down.

As far as such a project as Jamaica Bay is concerned, however, it may be of interest to engineers to know that in recent years it has been the wholesome policy of Congress to require greater local co-operation toward paying the cost of main channels and of such protective works as are necessary to insure the integrity of these main channels whenever the project to be undertaken is of a more or less speculative character, as is true, for example, in the case of Jamaica Bay where no commerce exists and where the character and extent of future traffic are, after all, more or less matters of opinion. It may well be that when in the future the larger project for making available the waters and shores of Jamaica Bay again comes up for consideration, Congress will feel justified in asking the community to bear its fair share of the risk, not only by improving real estate, but also by paying a fair portion of the cost of channels which in the end may possibly prove to be none too profitable.

Resuming the enumeration of the existing channels in the Port of New York, the Newark Bay Channel is 20 ft. deep and 400 ft. wide, with a side channel of similar dimensions connecting the central channel with the Port of New York development. At present a project is under consideration for deepening this channel to 30 ft. Some difficulty has been experienced in deciding as to the best policy regarding this particular channel because of the obstructive bridge near its foot, connecting the mainland with the Bayonne Peninsula. This bridge which belongs to the Central Railroad of New Jersey, is now required to be reconstructed under the existing project, but as yet no change has been made, and it is perhaps worth saying that until it is positively

known that this bridge will be changed to permit the easy passage of deep-sea carriers, it would be useless to spend any money in deepening the main Newark Bay Channel.

The remaining important channel is the Arthur Kill Channel, which is 25 ft. deep and 400 ft. wide. This channel is a very busy waterway and, recently, the traffic has been so heavy as to call for a re-examination of its needs. The War Department has recommended to Congress that a project for a channel 30 ft. deep and 400 ft. wide through Raritan Bay and Arthur Kill, to connect with the corresponding channel in Kill van Kull, be adopted at a total cost estimated one year ago (1920), at about $10 400 000.

This summarizes the situation as to the main channels in the Port of New York. The War Department and the Corps of Engineers constantly study and observe traffic and terminal conditions in order that they may be able to appraise intelligently the needs of the ports. Whenever these studies indicate the desirability of greater channel accommodations, either by deepening and widening existing channels, or by extending them and creating new ones, the need will certainly be reported to Congress in ample time to avoid the damage or inconvenience that might result from a failure to take timely action. So far as is consistent with the duty of protecting the National Treasury from unprofitable or unnecessary expenditures, the speaker can safely promise that the War Department will always, in the future as in the past, deal not only considerately but liberally with shipping and with water-borne traffic. When their needs are in the form of an extension of any channel so that additional terminals may be created to supplement existing terminals already fully used, no difficulty will be experienced as far as the Department is concerned in securing a recommendation to Congress that the channels be appropriately enlarged. It is necessary, of course, to safeguard this promise by the statement that its policy is to insist that proper use be made of the portions of the shores of any port to which access has been afforded by the creation of adequate channels, and this insistence may well take the form of requiring that where important sites are occupied by activities not located by necessity on the water-front, these sites be vacated and suitable terminals provided there before the Government is forced to further expense in the provision of channels. This weakness in the development of the Port of New York is well known and is emphasized in the monumental report of the Joint Commission in which attention is repeatedly drawn to the fact that, especially on the two sides of the Hudson River, the shores are used as railroad stations and not for transfer from rail to ship and ship to rail. This evil is especially noticeable on the west front of the Hudson, where practically all the Jersey City front and much of that of Hoboken and Weehawken is so used. What a relief it would be to the Port of New York if these considerable lengths of shore line were available for the construction of modern piers and their adjuncts. It ought not to be regarded, therefore, as unreasonable for the National Government to insist, as was recently done in the case of a South Atlantic port, that all available frontage be properly utilized before extensions of channels are made.

Reverting now to the relation of the War Department to the National port terminal problem, it is probably well known that, in recent years, Congress on frequent occasions, has indicated its view that port and river terminals were essential adjuncts of waterway improvements and that they should be incorporated in any proper plan of such improvements and required to be supplied by the local interests. The extent of the preoccupation of Congress with terminal matters and the importance which it attaches to the creation of proper terminal facilities is shown by the legislation incorporated in Section 7 of the River and Harbor Act, approved July 18th, 1918, which reads as follows:

"Section 7. That hereafter the Chief of Engineers, United States Army, shall indicate in his annual reports the character of the terminal and transfer facilities existing on every harbor or waterway under maintenance or improvement by the United States, and state whether they are considered adequate for existing commerce. He shall also submit one or more special reports on this subject, as soon as possible, including, among other things, the following:

"(*a*) A brief description of such water terminals, including location and the suitability of such terminals to the existing traffic conditions, and whether such terminals are publicly or privately owned, and the terms and conditions under which they may be subjected to public use.

"(*b*) Whether such water terminals are connected by a belt or spur line of railroad with all the railroads serving the same territory or municipality, and whether such connecting railroad is owned by the public and the conditions upon which the same may be used, and also whether there is an interchange of traffic between the water carriers and the railroad or railroads as to such traffic which is carried partly by rail and partly by water to its destination, and also whether improved and adequate highways have been constructed connecting such water terminal with the other lines of highways.

"(*c*) If no water terminals have been constructed by the municipality or other existing public agency there shall be included in his report an expression of opinion in general terms as to the necessity, number, and appropriate location of such a terminal or terminals.

"(*d*) An investigation of the general subject of water terminals, with descriptions and general plans of terminals of appropriate types and construction for the harbors and waterways of the United States suitable for various commercial purposes and adapted to the varying conditions of tides, floods, and other physical characteristics."

A further expression of the policy of Congress in regard to terminals is contained in the River and Harbor Act approved March 2d, 1919, in which the following statement is incorporated:

"It is hereby declared to be the policy of the Congress that water terminals are essential at all cities and towns located upon harbors or navigable waterways and that at least one public terminal should exist, constructed, owned, and regulated by the municipality, or other public agency of the State and open to the use of all on equal terms, and with the view of carrying out this policy to the fullest possible extent the Secretary of War is hereby vested with the discretion to withhold, unless the public interests would seriously suffer by delay, monies appropriated in this Act for new projects adopted herein, or for the further improvement of existing projects if, in his opinion, no water terminals exist adequate for the traffic and open to all on equal terms, or unless satisfactory assurances are received that local or other interests will

provide such adequate terminal or terminals. The Secretary of War, through the Chief of Engineers, shall give full publicity, as far as may be practicable, to this provision."

It will be noted that the Chief of Engineers is required to report annually on the character of the terminals existing in every harbor or waterway under improvement by the United States, and to express an opinion in regard to their adequacy. This requirement, of course, cannot be fulfilled unless consideration is given not only to the character and number of the terminals themselves, but also to the place of each port in the National transportation system, its tributary territory, the traffic it ought to serve, its probable future growth, and the effects, restrictive or otherwise, of existing channel developments. These questions are so important and far reaching that to answer them satisfactorily it has been found necessary to expand greatly the functions of the Board of Engineers for Rivers and Harbors which, until about two years ago, had for its main function the review of projects, active or proposed, for river and harbor improvements. At present, therefore, this Board, under a plan approved by the speaker, is making a close and intensive study of every important seaport in the United States and an analysis of its most useful place in a properly conceived National system. These investigations permit replies with considerable certainty to questions relating to the adequacy of port terminal development.

The special reports called for in Paragraphs *a, b,* and *c* of the 1918 legislation have been consolidated in a single report.* This bulky volume is in greater part a compilation of facts relating to all water terminals on waterways now being improved or maintained by the United States. Its principal service to the public, and especially to engineers, is to place before them in authoritative form the many things relating to existing port improvements, which usually are known only in a general way, if at all. For example, it gives to every careful reader a serviceable idea regarding the state of development concerning any port in which he may be interested.

The investigation of the general subject of water terminals, called for in Paragraph *d* of the 1918 legislation, has been made by the Board of Engineers for Rivers and Harbors, and the results are contained in a comprehensive report† by Capt. F. T. Chambers, U. S. N., associated with the Board as expert adviser on terminal matters. The illustrations accompanying the report are numerous and afford examples of nearly every useful type of terminal known to exist. In order to show the scope of this report, the speaker will enumerate the headings, as follows:

Ocean terminals are considered under the following sub-divisions: sheltered location necessary; ports; terminal facilities; tides and floods; anchorage; fore and aft moorings; locks and basins; docks, quays, wharves, and piers; open or uncovered terminals; ground storage for materials in bulk; ocean and inland waterways terminals; other open or uncovered terminals; naval stores; covered terminals for material in bulk; storage for liquor material in bulk; fire protection; grain elevators; covered terminals for miscellaneous or package freight; types of construction; piers; materials for construction;

* House Document No. 652, 66th Cong., 2d Session.

† House Document No. 109, 67th Cong., 1st Session.

examples of pier construction; quays and wharves; warehouses; warehouses for special purposes; railroads; lighterage; mechanical equipment; ship's gear; ship's gear in combination with portable winches and cargo masts; wharf cranes; types of cranes; monorails and telphers; conveyors; slot conveyors; pan and bucket conveyors; chain hook conveyors; portable chain and belt units; gravity conveyors; gravity roller conveyors; piling; machines; fixed elevators; trucks; self-propelling vehicles; tractors and trailers; floating equipment; floating derricks; steam lighters; suction grain elevators; chain belt bucket units for grain; bunkering equipment; portable bunkering units; car floats; repairs and reconditioning; types of dry docks; seaport terminal; planning, including harbors, channels, ship characteristics, tides, available water-front, character of exports and imports, open and covered wharfage, railways, piers, and slips, floor space under cover, railway trackage at water-side, hoisting equipment, railway tracks and roadway at rear of shed, railway yards, disposal of dredged material, quay or wharf construction storehouses, mechanical equipment, justification for transit sheds of large area, port terminal charges; port terminal control; forms of port control, public, private and railroad, railway port terminal charges, public management.

Inland waterway terminals are considered under the following subdivisions: canals and rivers; navigable depths and variation of water level; terminals for waterways of constant level; terminals for handling bulk cargo; terminals for rivers of wide variation of water level.

The 1919 legislation as to the essentiality of having at least one public terminal at each waterway locality under improvement by the United States, is a plain declaration of a belief which has long been held in public places, although not so formally expressed. The idea is sound, but does not go far enough. Every one who has come in contact with the port terminal problem sooner or later has been forced to conclude that perhaps the greatest evil in the situation and one of the principal "National Port Problems" is the undue extent to which the water-fronts of the principal seaports have passed into railroad control or ownership. The effect of such railroad predominance has often been to influence the prosperity of the seaports, some of which have been favored at the expense of others. The situation in this regard is quite fully covered by the Board of Engineers for Rivers and Harbors, in a recent report which relates to the South Atlantic and Gulf Ports, and recommends steps to prevent the railroads from continuing this kind of activity, by compelling them to make rates for terminal services fairly compensatory for the work involved and the use of the facilities concerned. The belief of the Board is, however, that the railroads should be compelled to divest themselves of their ownership of these port terminal properties and that preferably they should pass to public ownership and control.

It may be of interest to quote the closing paragraphs of the report on the investigation of terminal charges at South Atlantic and Gulf Ports by the Board of Engineers for Rivers and Harbors:

"In promoting water transportation, the War Department and the Shipping Board recognize that a primary necessity is the provision of adequate terminals, and they have an interest in securing such changes in existing conditions as may be necessary to establish them. The absorption of terminal charges in the rate for the line haul and their inadequacy render it impracticable for the private terminal to compete with the railroad terminal for through business just as a large mercantile establishment by selling one commodity below cost can destroy the business of a special dealer in that commodity, and still make

a profit on the total business. It is well known that except at a few places where unusual conditions prevail, private terminals cannot be operated at a profit. The situation is somewhat different in respect to publicly owned terminals. Municipal corporations generally have good credit and are able to borrow money at low rates. Even though public terminals may show an apparent deficit, it may well happen that they will cause such growth in general prosperity and in the volume of business as will outweigh any tax burden caused by the terminals.

"To permit the free flow of commerce through our ports, the obstacles in the way of creating modern terminals must be removed, and since the carriers themselves cannot be expected to initiate the necessary reforms, prompt steps should be taken by the United States. Two remedies have been suggested: First, a scale of terminal charges sufficient to cover the actual cost of the service, plus a reasonable return on the investment. This would enable private terminals to operate if the railroads were obliged to pay them such charges for services actually performed. Second, the discontinuance of the practice of absorbing terminal charges in the rate for the haul. The latter remedy has been widely advocated, but clearly presents some difficulties in its application, and necessarily implies a complete revision of all rates to and from water points.

"In some cases an effective remedy would be the ownership and operation by the State or municipality of all water terminals used for public transportation purposes, including a belt-line railroad affording connection with all wharves and with all railroads serving the port. With the switching, wharfage, handling, and storage charges, in the hands of the State or municipality, every terminal within the port might expect equal treatment, and the responsibility for providing adequate facilities would rest squarely upon the community itself. This remedy is not practicable at many localities, however, owing to the extensive occupation of the water-front by private interests.

"The difficulties confronting the solution of the problems herein considered are fully recognized, and it is obvious that decision as to the wisdom of any plan for correcting existing conditions should be reached only after the most searching investigation of its effect upon the movement of traffic and the relationship between competing ports. The matter is of such importance to the commerce of the country and the success of the merchant marine, however, that no difficulties should be allowed to stand in the way of securing an adequate remedy."

On March 25th, 1921, the Secretary of War brought this situation as to inadequate or unfair terminal charges to the attention of the Interstate Commerce Commission. As this letter is an impartial summary of the situation, it seems of sufficient interest to be read in its entirety, and is as follows:

"By reason of the war emergency the country has been provided with a large commercial fleet upon which a very large sum already has been expended. It is indispensable to our safety and prosperity as a nation that this fleet operate successfully and economically in competition with the vessels of our maritime and commercial rivals.

"Indirect and direct charges of a ship in port run from $1 000 to $5 000 per diem, and every day spent by each ship in port in excess of the minimum time required to load and discharge cargo, and to take on fuel and supplies, is a waste, which summed up for the entire fleet amounts under present conditions to millions of dollars per annum. Ships earn money only when they are kept moving with profitable cargo. Efficient operation, therefore, requires that the days spent in port be the fewest practicable.

"With the object of promoting, encouraging, and developing ports and transportation facilities in connection with water commerce, Section 8 of the Merchant Marine Act authorizes the Shipping Board, in co-operation with

the Secretary of War, to investigate territorial regions and zones tributary to ports, in order to determine what rates, charges, rules, or regulations should be established, and, in case changes are found necessary, to submit its findings to the Interstate Commerce Commission for such action as the Commission may consider proper under existing law. The Board of Engineers for Rivers and Harbors on behalf of the War Department has therefore been investigating the subject of port facilities, and both the Chief of Engineers and the Board are convinced that the success of our commercial fleet is largely dependent upon the economical operation of our ports.

"The results of the above investigation have been compiled by the Statistical Section of the Board of Engineers for Rivers and Harbors and are shown in the attached paper, which also contains excerpts from the various laws assigning to the War Department and the Shipping Board duties in connection with port regulation and port development. In view of the evidence presented in this compilation, it is requested that the subject of port charges and practices at South Atlantic and Gulf ports be considered with a view to the initiation of the necessary reforms, so that commerce may flow through our ports with the greatest possible speed and the least friction; or, in other terms, at the least expense to both the shipper and the ship owner.

"The Chief of Engineers points out that there are three forms of terminal control. These are (*a*) public (State or municipal); (*b*) railroad; and (*c*) private; and at none of our important ports is the control absolute under any one of these three heads. San Francisco and New Orleans have the nearest approach to complete public control, but at all of our important ports there is some railroad control, and at many of them there are also private terminals. Under present conditions of railroad operation, the railroads have no direct incentive to build or to operate terminals such as would turn the ships around in the least practicable time, and, even should such incentive exist, they are not now financially able to provide the costly wide piers and the expensive mechanical equipment necessary to the greatest economy. As will be seen * * * the tariff charges for handling cargo over the railroad terminals at practically all of our South Atlantic and Gulf ports are from one-quarter to one-half the actual cost of performing the work and this is the amount that the railroads allow private terminals for such service performed by them. Under such circumstances private terminals now in existence do mostly a warehousing business, or handle local freight only. They cannot exist on the railroad divisions for through overseas business. Private capital cannot therefore be attracted to invest in such terminals, nor can the largest cities be expected to construct modern terminals, if, as sometimes happens, the situation created by the railroads is such that it is certain in advance that an adequate return cannot be had with which to amortize the bonded debt and that the indirect gains are so seriously restricted as not to counterbalance this direct loss.

"Obviously, it would be advantageous to the public to have full freedom in the choice of routes over which its commodities may be transported. Such choice is now, however, limited and one of the limiting conditions is the fact that the route that otherwise might be preferable may have at the terminal port facilities such as wharves, transit sheds, storage warehouses, and the like whose capacity is merely sufficient to care for existing business or whose equipment may be poor, antiquated, inefficient, or uneconomical. Such restraints upon the adoption of the economic path for moving traffic should, in the general interest, be removed and the opportunity should be afforded all concerned to create, with reasonable prospect of adequate return, such port terminal facilities as seem needed and desirable. The only way in which this can be brought about is to put all such terminals upon a proper basis. The difficulties of the problem are no doubt considerable, but it is believed that this can be done, in some cases by revising the terminal tariffs so as to be reason-

ably compensatory for the service actually performed, thereby permitting privately owned terminals to be created and operated. In general, however, the ownership and operation of all terminals by the local public authorities or by the State constitute the best solution and this course should be adopted wherever possible.

"I would ask that the facts herein presented be considered with a view to the early application of such remedies as may promote the end in view, namely, the reduction to a minimum of the cost of transporting the goods that enter into our water-borne trade, both foreign and domestic."

As a result of these representations the Interstate Commerce Commission has held a series of hearings at all the ports included in the investigation, the first having been held at Norfolk, Va., on September 9th, 1921. It is not unreasonable to expect that the final outcome will be to place the port terminal situation as to charges on a sounder basis.

It will be observed that the War Department and the Corps of Engineers believe it to be for the best interest of the public that water-front improvements be publicly owned and operated, but in this matter of public operation it is important that caution be observed not to permit such operation by the political agencies sometimes utilized by city and State governments, which would be to the serious detriment of seaport terminals. The form of public operation which the War Department favors and which the speaker strongly advocates is that under which some form of port authority exercises control on behalf of the public, similar to the English so-called "Port or Harbor Trust." This body is dominated by representatives of the shipping, commercial, and business interests of the port and the purely political members constitute a small minority.

One of the greatest obstacles to the progress of American shipping business and to the promotion of the export and import traffic is the lack of unity of management. In this respect, Americans should take a leaf from the book of their British competitors, who have made this subject a study for many years of successful maritime business. Every considerable British port has a really unified management.

Forms of Port Control.

In general, it may be said that port terminal control is of three kinds: (1) public; (2) railroad; and (3) private.

(1).—*Public Control.*—This is best illustrated by the practice of such ports as Hamburg, which is a commercial State, and of London, Liverpool, and Bristol, England, which more nearly represent what we should endeavor to attain in the United States. The Port of London suffered for many years from the internal competition of a number of separate dock companies within the port. It was not until July, 1908, that the entire port property was taken over and put under the London Port Authority, competition within the port thus eliminated, and the port as a whole put in a position to direct its energies to competition with other ports. This Port Authority is called in England a "Trust", not as this word is generally used in the United States, but in the sense of a corporation operating without profit for the public benefit. This does not mean that port charges are not assessed, but only that the revenues

obtained from such charges are devoted exclusively to the interest of the port terminal, and not, as in the City of New York, to such extraneous matters as the cleaning of the city streets and other items of city maintenance. In other words, where New York devotes a considerable part of its revenue from the city-owned piers to general city purposes, London devotes its tolls to that part of the welfare of the community which is bound up in the promotion of its shipping trade. Another organization called "The Trinity House" is responsible for the lighting and buoying of the river. The Metropolitan Police are the guardians of the port, and the Corporation of the City of London supervises the sanitary conditions with regard to shipping.

The Port Authority is empowered to collect tolls on ships and goods, and has full jurisdiction over the various docks and terminals. It is constituted as follows: Appointed members, 10: one by the Admiralty; two by the Board of Trade; two by the London County Council (being members of the Council); two by the London County Council (not being members of the Council); one by the City Corporation (being a member of the Corporation); one by the City Corporation (not being a member of the Corporation); and one by Trinity House. The elected members number 18: Seventeen by the payers of dues, wharfingers, and owners of river craft; and one by the wharfingers. In addition, the Chairman and Vice-Chairman may be appointed from outside the membership of the Authority, making the total possible number 30.

The organization at Liverpool is known as the Mersey Dock and Harbor Board. It is quite similar in its constitution to the London Authority, and has 28 members; 24 of whom are elected by the dock rate-payers, that is, persons paying rates and dues on ships and goods only, the remaining 4 being appointed by the Mersey Conservancy Commission, which consists of the First Lord of the Admiralty, the Chancellor of the Duchy of Lancaster, and the President of the Board of Trade. The Board of Trade, unlike the Boards of Trade in the United States, is a National institution corresponding in many respects with the Department of Commerce. In making appointments to the Mersey Board, the effort is made to give the various trading interests in the local shipping community a proportionate representation, and at the present time this is said to work well. Such organizations as those at London and Liverpool may seem to the casual reader unwieldy, but each organization is divided into separate working committees with apparently good effect.

The Port of Bristol is owned and operated by the Municipal Corporation, which thus has jurisdiction over the entire dock system, the control being vested in the City Council, which, for purposes of administration, appoints a subcommittee. This committee employs the executive officials and gives them a free hand in the management. It is declared that politics plays no part in this administration.

(2).—*Railroad Control.*—Some of the ports of England, notably, that of Immingham, are entirely in the hands of the railways. The port terminal is operated as a part of the railway system, and as a means of getting business. In fact, many ports in the United States are operated on this same railway basis, the port terminal charges being nothing, or nominal, wherever the railway is enabled to get business by offering dockage to the ship.

(3).—*Private Control.*—Terminals of this class are operated by private individuals or corporations in the pursuit of their own business in all parts of the world. They may be adapted to the handling of only one product, or to miscellaneous freight. The manufacturing concern may be located at the terminal, and may use its water-front facilities in direct connection with its business only, or, in some instances, may conduct a separate terminal and warehousing business.

General.—It may be said in general that taken as a whole no port in the United States comes under any one of the three previously named headings. In all ports of any size all three methods of administration are in force. It is true that at San Francisco and New Orleans public port authorities are controlled by the State, but, at San Francisco, the Harbor Board confines its activities to the water-front at San Francisco, while the railways have terminals across the Bay at Oakland, on which side of the harbor there are also private terminals. At New Orleans, also, the State Board and the railways have separate terminals.

In some ports it has been the practice to lease the publicly owned terminals to private operators. Although there may be some financial justification for this policy, it seems well to observe some degree of caution in granting exclusive and particularly long-time leases for the use of piers and wharves. The recent experience has been that such exclusive leases may result in less efficient use of the terminals than is desirable for the best interests of the port. A lessee, for example, may keep his pier idle rather than permit its use by an actual or potential competitor, or he may make such high charges as to drive business away from the port. This actually happened in New York during the World War. If leases must be made, they should preferably be in the form of a first call on a berth, leaving to the public port authority the right, when the berth is vacant, to assign to it vessels other than those belonging to the lessee.

At the same time, the prestige of a port depends greatly on the number of regular services that it affords. Liners belonging to such services are most readily identified by the public when their sailings take place from definitely known piers. The system of first call on a berth usually meets this situation adequately, but if it is necessary to promote liner service a limited number of concessions from the rule might be made.

It seems well to repeat the result of the review of the port situation in the United States, as stated in the report of Capt. Chambers, to which reference has previously been made. This review shows:

1.—That there is not, in most ports, a well co-ordinated management and that a well constituted port authority is the first need.

2.—That under these port authorities, comprehensive plans should be evolved, based on principles hereinbefore laid down.

3.—That a port can only be successful when this plan is based on the business available in its tributary area and brings ample railway facilities into the closest practicable juxtaposition with the water-front, with sufficiently wide areas available for cargo classification.

4.—That, with the increased cost of labor, mechanical means, wherever practicable, should be adopted for handling goods, and that such means should be at hand for every kind and shape of package.

5.—That ample railroad tracks should be available close to the terminal for car storage and car classification.

6.—That ample warehouse capacity should be provided, in order that both ships and cars may be dispatched in the shortest practicable time. Too many American ports are lacking in warehouse facilities, or have such facilities at an inordinate distance from the terminal, thus involving cartage, or extra handling, for local railroad haul.

7.—Where cartage is necessarily a feature in the port business, roadways and loading platforms should be provided for the full accommodation of trucks.

8.—That bunkering facilities of such character as to supply the necessary fuel to the ship while handling cargo should be available.

9.—That ample repair and dry-docking facilities should be provided.

No matter how ample the provision of terminal piers, sheds, railways, and mechanical equipment, a port will only succeed under an efficient and well co-ordinated management.

Although this discussion of the general subject of "Terminals" has been from the point of view of the War Department, the speaker feels that in closing he should state that it is the policy and the wish of the Department to be helpful to the people it serves, and in the discharge of the trust imposed and in using the discretion confided by law, the desire is to be liberal and not narrowly technical. Therefore, an invitation is extended for the frank and full discussion of problems connected with the subject, with the promise that they will receive patient and sympathetic consideration.

DEVELOPMENT OF THE SMALLER PORTS

By Frederic H. Fay,* M. Am. Soc. C. E.

As a nation, Americans are slowly coming to a realization of the economic value of their seaports to the country, a lesson learned long ago by their European neighbors and more recently by their Canadian cousins who have been more alert than themselves and by whose example and foresight they may well profit.

Before the World War it was commonly stated that a ton of freight could be hauled 5 miles by water as cheaply as it could be carried 1 mile by rail. With changed conditions, due to the war, this ratio may now be changed. Leaving aside, however, for the moment, the cost of loading and unloading which admittedly constitutes a large part of a transportation charge, it is to-day unquestionably true that freight can be hauled at a given cost a far greater distance by water than by rail. As a nation, Americans should be giving more serious consideration to the importance of water-borne, coastwise traffic, and since the opening of the Panama Canal, particularly to the possibility of greatly increasing coastwise traffic between the Atlantic and Pacific ports. A full development of this domestic commerce by water routes between these two coasts should result in extensive changes in the routing of freight by rail and in the rail-rate structure, stimulate the interchange of commodities between remote sections of the country, and prove of economic benefit to the nation as a whole. Although only in its infancy, the small beginnings which in these abnormal times have already been started in the Atlantic-Pacific coastwise trade, have had a marked effect in certain directions. Low coastwise rates have been established which, in some instances, are less than half the transcontinental rail rates, and the results thus far give promise of the larger results to follow when business has returned to normal conditions.

Until recently, the development of American ports has been left very largely to private interests, usually to the railroads which naturally have made their developments with a view to their own immediate gains and without regard to the larger advantages to the public as a whole through the unification and co-ordination of the development of a port as a single unit. The American public is gradually awakening to an appreciation of the importance of public ownership or control of port facilities and of the commercial value of ports, especially as agencies in foreign commerce. Efforts toward public port development should not be concentrated on a few of the larger ports and primarily for purposes of foreign commerce; public attention should also be drawn to the benefits to be gained by the development of the smaller ports as well, not having in view foreign commerce alone, but also the need of stimulating water-borne domestic commerce and the economies and benefits resulting therefrom. That part of the Engineering Profession concerned in port-development work has a public duty to perform in giving serious study to the situation as it exists at the smaller ports and of acquainting the public with the facts as they find them.

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Economic Value of the Seaports.

A community located on a busy harbor enjoys many special advantages. Marine carriers make large capital outlays in connection with their terminal operations. They buy coal, provisions, and other supplies from local dealers; pay substantial sums in wages to longshoremen and freight-handlers; spend money for repairs and employ towing companies and pilots. Even at many of the smaller ports millions of dollars attributable to port activities alone are distributed annually in the port community.

In a larger way, however, the port is the servant of the interior. For every ton of local freight shipped to and from the port community itself, there are usually many tons shipped to and from the district immediately tributary as well as from more remote inland territory. This indicates the true relationship between the seaport and its hinterland. The port serves the interior in the latter's dealings with coastwise and foreign ports, and provides the smoothest mechanical and commercial means for the movement of inland freight to and on the water. Its development calls into life new water lines and betters the service of existing lines, and its merchants find new markets for inland products.

A well-developed port is of indirect benefit to city, State, and nation, and its development should not be left in private hands, but should be under public control and participated in jointly by municipal, State, and Federal governments.

The principle of Federal aid has long been established through the assumption by the Federal Government of the work of dredging and improving harbor channels, harbor protection, and the provision of aids to navigation.

More recently State agencies have undertaken the building and development of water-front terminals, the reclamation of water-front lands, and even the dredging of harbor channels, to supplement the improvements made by the Federal Government.

In a few instances municipalities, at their own expense, have provided extensive water-front terminals; in others, they have co-operated with State agencies in such undertakings, as, for example, in acquiring water-front sites on which State funds are spent for pier construction.

The public is coming to a general recognition of the soundness of the principle that the seaports are an important National asset and, as such, should be under public, not private, control; and, further, that Federal aid should properly be given to the improvement of the water facilities of a harbor, while States and municipalities may properly work together in providing water-side facilities and necessary improvements on land.

Requisites of a Port.

It may not be out of place to consider briefly the chief requisites of a port, since the engineer must have these fundamentals in mind when studying the problems of any port, large or small.

Water Facilities.—There must be a good harbor, well sheltered, and free from ice; a natural harbor, if possible, or, at least, a location such that an harbor may be developed at moderate expense. The harbor should have

natural deep-water channels, or else a bottom such that adequate channels may be dredged and maintained at reasonable costs, and should preferably be in close proximity to the open ocean with a safe and direct approach channel and, if possible, an absence of an outer bar.

Rail Facilities.—On land, there should be adequate railroad facilities, not only locally throughout the district immediately tributary to the port, but also trunk-line connections to more remote points in the interior of the country. At the port itself, there should be adequate railroad terminals, including classification and storage yards and supporting yards in close proximity to the water-front piers. Important for the success of a port are belt-line rail facilities for the interchange of freight between the several roads and for bringing freight from each railroad to each pier, and fortunate is the port where such facilities exist or can be provided at reasonable expense.

Pier Facilities.—The pier is the connecting link between rail and water carriers. Piers should have transit sheds sufficiently large for the assembly and rapid handling of cargoes. Present-day ships call for piers of large size and larger shed capacity than those commonly built in the past. There is general agreement as to the importance of railroad connections to piers, but many port authorities, shippers, and steamship officials in the United States have been slow to grasp the importance of equipping piers with adequate, modern, mechanical apparatus for the rapid handling of freight.

In addition to the general factors already noted, recognition should be given to the importance of good highways throughout the territory immediately tributary to the port. The motor truck has its legitimate field in the inter-urban transportation of high-class package freight, and by its use many inland industrial communities may have connection with the seaport more cheaply than by rail.

It is obvious that a port situated at the mouth of a navigable stream has the added advantage of water transportation as well as rail connection with its hinterland.

Where advantages such as those already outlined exist, public authorities should give serious consideration to the possibilities of their proper utilization, and this is particularly true of many of the smaller ports. Europeans have realized more keenly than Americans the importance of access to the sea, and, in many instances, enormous sums have been spent in overcoming natural obstacles and in making seaports out of inland, or semi-inland, cities, as at Manchester, Glasgow, Amsterdam, Rotterdam, and Hamburg.

It is in connection with the smaller ports particularly that the engineer should live up to his duty as an economist, study each local situation with critical analysis and breadth of vision, and be a pioneer in calling public attention to the possibilities that lie before us.

Danger of Concentration in a Single Large Port.

In the United States the tendency has been to foster the development of the Port of New York which, for years, has handled approximately 50% of the country's foreign commerce, and ignore the natural advantages of, and the importance of developing, many of the smaller ports. Americans may well

ask themselves the question whether it is a wise policy for the country as a whole to have so large a proportion of its commerce tied to a single port.

Less than two years ago, the Collector of the Port wrote of conditions at New York* as follows:

"We are accustomed to speak of New York as the greatest commercial port in the world. So it is. To-day its water-borne commerce, foreign and coastwise, is greater than that of any other two ports in the world; but that is no reason why it will continue to hold this place for all time. * * * When the pressure of war traffic came upon us the port of New York 'broke down', or so nearly broke down that it was a matter of National distress and alarm. Along the extensive shore line of the inner harbor there was ample space for handling all the traffic the war sent to our port; but only a part of the harbor shore line was available. There were not piers enough to accommodate the vessels, and most of the available piers were too small, inaccessible, improperly equipped, or being used for some purpose necessary for the domestic demands of the city."

In war time, the successful blockading of this one port by a strong enemy fleet would be a tremendous handicap. At any time, local conditions may precipitate a strike of longshoremen, or freight-handlers, with paralysis of shipping, as New York well knows by recent experiences. Only last winter (1920–21) the influx of immigrants at New York was so great that the immigration and quarantine facilities of the port were unable to handle them, and it was necessary to divert ships to other ports to land such passengers. The cost of handling freight is enhanced by lack of belt-line railroad facilities, by the necessity for lighterage to supplement the roads, and by lack of modern piers equipped with modern freight-handling facilities. Furthermore, the values of New York water-front property are so great as to place a substantial over-head burden in the carrying charges on the investment in water-front terminals. All these factors tend to enhance the cost of handling freight in New York and the question may well be asked whether a considerable proportion of the business now passing through that port could not be handled cheaper, with consequent economic saving to the nation, at some of the smaller ports.

Efficiency and economy must be applied to the nation's business if it is to take its full part in world affairs.

Advantages of Development of Smaller Ports.

Instead of putting so many "eggs in a single basket", which may come to smash at any time, as has been proven at New York during and immediately after the World War, is it not better to get a diversion of commerce through other ports possessing good, if not equal, natural advantages, and to awaken in the public mind a realization of the National, State, and local benefits, arising from suitable public development of some of the smaller ports?

The development of smaller ports means a diminution of labor troubles and less likelihood of strikes having such serious effects on water-borne commerce. Such development means not only the building up of cities where the ports are located, but also the districts tributary thereto and a more even

* "New York's Endangered Commercial Leadership," by Byron R. Newton, Collector of the Port of New York, *The Street*, December 31st, 1919.

distribution of prosperity throughout the country at large. Americans have been too much inclined to consider port developments only in the light of foreign commerce, and have ignored the advantages of port development in increasing and cheapening domestic commerce and in lessening the cost of living. In the smaller ports, the cost of development is substantially less than the provision of equally good facilities in a large port like New York. Water-front properties are relatively cheap. Belt-line rail connections are usually more readily provided, if they do not already exist.

To-day, large industries are coming to realize the disadvantages of concentration in the larger cities and the advantages to be gained by locating their plants in smaller communities where better living and labor conditions are found. The development of smaller ports to cheapen transportation to and from smaller communities, both on the seaboard and in the district tributary to the port, is of direct concern to industries which are seeking to develop plants in the smaller cities and towns. This tendency toward distribution rather than concentration is a helpful one for industries and the country as a whole, and makes necessary more than before serious consideration of the need of development of smaller ports to handle not especially overseas traffic, but, primarily, coastwise traffic.

Port Development at Public *versus* Private Expense.

The proper development of American ports is to-day a vital problem. With only a few exceptions, ports in the United States were originally developed by railway or other private interests, and there is ample proof that, in most cases, the individual interests have failed properly to develop and co-ordinate the water-front facilities. Usually, the development has been narrow and selfish, and each railroad naturally has looked after its own individual interests. The recent trend of development has demonstrated plainly that the port must be considered as a whole, with the railroad and other terminal facilities co-ordinated so that they develop the port and the territory tributary to it in the most economical manner and on the broadest possible scale.

Where port development has been by railroad agency alone, it has naturally progressed slowly and, in general, only with the assurance that it would prove to be a paying investment. The tendency until recently has been for each railway to attempt to secure for itself the most advantageous location for its wharves and docks, and then to throttle the development of the port as a whole by setting up artificial barriers through the medium of switching charges, absorption of wharfage and handling charges, etc. As a result, many ports have become in reality a collection of several small ports, each serving as a terminus for an individual railroad, interchange of traffic between various parts of a harbor being rendered both slow and expensive.

A study of North American ports where such facilities are publicly owned or controlled shows that such ports have been a success in every case and this is obviously true for some of the notable ports of Europe. Not only has the operation of these terminals proven the investment to be sound, but the ports have been distinctly benefited by public control. Discriminative railroad practices have been eliminated, more flexibility of operation has been assured,

and commerce has been increased, due in large measure to the greater opportunities for expeditious and cheap handling of traffic.

No private party can be expected to make the large expenditures required for adequate port facilities. These must be made by the public, since States and cities can command a lower rate of interest and will benefit not only from a direct return, but even more largely from the indirect return which increases the business and the prosperity of the city and State in which the port is located. Private parties must consider the cost of water-front terminals purely from the standpoint of direct and immediate financial return, sufficient not only to pay carrying charges, including depreciation, but to return a reasonable profit to the investors. On the other hand, the community may very properly find the greater part of its profits in indirect returns, such as the prevention of the port from being throttled by its more active competitors, increase in commerce, and the promotion of the welfare and prosperity of the city and of the State at large.

Portland, Me., as Typical of the Smaller Ports.

Portland, Me., is a good example of one of the smaller but important ports the development of which has long been neglected, but where public sentiment has been aroused. The needs have been recognized, and the State and the municipality are co-operating toward a modern development. Portland possesses a harbor which, from the standpoint of natural advantages, is one of the best in the United States. Although not comparable with that of New York in size, it is of ample area with a natural deep-water channel and with no bars at its entrance. The harbor is well sheltered, in close proximity to the open ocean, with a channel so direct that steamships making regular calls dispense entirely with pilots and enter the harbor at any time of day or night, at any season of the year, under the direction of their own officers.

Portland is the Atlantic terminus of the Grand Trunk Railway and is the natural winter port for the Dominion of Canada. It is also a terminus of the Boston and Maine and the Maine Central Railroads, and is the nearest port in the United States to the United Kingdom and Europe. That the Federal Government has considered it to be one of the important Atlantic ports is shown by the fact that the harbor has been heavily fortified. During the World War, troops and supplies were shipped therefrom to the full limit of the existing water-front terminals.

Up to the present time, the only piers accommodating overseas shipping were those of the Grand Trunk Terminal; and, except for certain other wharves owned mostly by railroads and equipped for handling such bulk freight as coal, china clay, and sulphur, the wharves are obsolete and relics of the days of sailing ships when Portland had an extensive trade with the West Indies.

Recently, however, the need of increased water-front terminal facilities has been keenly felt. Certain steamship lines which have sought to establish themselves have been unable to do so on account of the lack of facilities for the accommodation of their ships. Only last winter (1920-21), the North Atlantic and Western Steamship Company, the boats of which, engaging in

the Atlantic-Pacific coastwise trade, had been berthed during the summer at the piers of the Grand Trunk Railway Company, found great difficulty in securing accommodation during the winter months, at which time the Grand Trunk Terminal is used to capacity, and succeeded in placing only a few boats at one of the Railway Company's piers through special arrangement with that road and because the Company's business last winter was not at a maximum.

Through the instigation of the Portland Chamber of Commerce, an agitation was begun which has resulted in the starting of further development through co-operation of city and State. The city has just provided the site of the first of a series of publicly owned piers, and the State is now about to construct such a pier alongside the Grand Trunk Terminal. Unlike many other ports, Portland already possesses a belt-line railroad connecting all the railroads entering the city, and the new State pier is located on this belt line so that freight at the pier will be handled on equal terms to and from the railroads. The State and the city are jointly embarking on a policy of port development with the idea not only of providing facilities for increased coastwise business, but also of increasing overseas commerce. The Maine Central Railroad connects with the Canadian Pacific Railway of Canada, which road it is understood has for some time desired entrance into Portland, and this entrance will be assisted by the building of the State-owned terminal.

Portland possesses a location which is unique among the North Atlantic ports in that through the Grand Trunk and Canadian Pacific Lines, it is a natural outlet, especially in the winter months, for territory extending as far as the Canadian Northwest; and because of the United States connections of these two roads, it is a logical outlet as well for the midwest territory of the United States bordering on the Great Lakes. Considering that the present tendency of Federal regulation of the railroads within the United States is to avoid competition in rates and to permit only competition in service, and, further, that Portland is served by Canadian lines which are not subject to regulation by the United States Government, it would seem as if it had possibilities which no other Atlantic port in the United States possesses, for competition in rates and the securing of business.

It is believed by the people of Maine that the development of this port will stimulate the industrial and agricultural growth of the entire State, especially since the State has available a large amount of undeveloped water power which, in these days of high priced coal, may be economically utilized to provide cheap power for industrial uses.

Administration of this new publicly owned terminal the construction of which is to be begun in the latter part of 1921, is in the hands of a board known as the Directors of the Port of Portland. The Board is composed of five members, one of whom represents the city, the others representing the four Congressional districts of the State. The people of Maine, and the Directors of the Port of Portland in particular, keenly realize that their obligations do not end with the provision of adequate, modern, water-front facilities, but that when these facilities are provided and before their completion, active steps must be taken to sell the port to their community and to the country at large.

RELATION OF WAREHOUSES TO PORT DEVELOPMENT

By M. A. Long,* M. Am. Soc. C. E.

The speaker will confine his remarks to his twenty years of experience in railroad work and his study of the warehouses of the different railroad companies, with relation to their inland, lake, and terminal ports.

For instance, at Cincinnati, Ohio, there is a railroad warehouse practically ¼ mile long. Why did the railroad company build that warehouse? Because, if it had not, it would have been compelled to enlarge its yards and buy more cars, using those cars for warehouses in which to store merchandise. By building that warehouse the company saved the expense of additional cars and yards, but a better reason for its construction was to attract business to the line which it had not enjoyed before. The merchants along the line of that railroad also store in this warehouse—advertising it as their own, for that matter—and then ship in small quantities from it to their customers along the line, or along other lines in the adjacent territory. The construction of that warehouse saved these manufacturers the necessity of building warehouses of their own in which to store the surplus which, particularly in seasonal business, they usually have.

If this is true at Cincinnati, is it not true of every seaboard port? Anybody going through New Jersey, and seeing the number of cars on the railroad tracks there, cannot but wonder why the railroad companies do not have warehouses in which to store goods, instead of storing them in cars which are expensive and while being used as warehouses do not earn any revenue, but show a loss. Each year, the railroad companies are compelled to buy more cars and build more tracks to keep up with expanding business. The remedy for this, and this applies to every port, is the construction of a series of warehouses. The speaker is quite sure that if they were built, the manufacturers of this country would use them and so would foreign merchants.

If a firm in any foreign country, doing business in America, had a surplus of goods, knowing that it could dispose of them to advantage in America, and knowing also that there were warehouse accommodations at one of the terminal ports, it would be to the advantage of that firm to ship them, using the warehouse company as its agent for local distribution; or, if its business was seasonal, ship to the warehouse to hold until it had proper opportunity to market its product. There will be a great deal of business of that character as soon as warehouses are built. Of course, there is a reason for all things, and the demurrage rates charged, or rather not charged, are the factors governing the warehouse situation at the ports. The following indicates some of the comparisons between rates charged on foreign and domestic freight. Let a firm ship a carload of merchandise to an inland port or city, after 48 hours' free time demurrage will have to be paid at the rate of $2 per day, for 4 days, and $5 per day thereafter. Suppose that same carload was being shipped to London: if it is shipped to New York on through bill of lading, 15 days' free time are allowed; if it is shipped on con-

* Pres., The M. A. Long Co., Baltimore, Md.

signment 10 days are allowed; for the first 20 days after the free time expires, 1 cent per 100 is charged, and for the next 10 days 3 cents per 100, and it is understood that a lower rate is now being put into effect. Thus, it is evident that, while a local merchant can well afford to pay for handling and storage warehouse charges, one dealing in foreign business does not have to pay such charges, because it is cheaper to use the railroad car as his warehouse.

It has been stated that it costs more to handle a ton of freight in the New York terminal than it costs to transport it by rail from New York City to Pittsburgh, Pa. That is true, and it shows that the railroad companies make their profits from transportation and not from terminal handling. If this statement is correct, the railroad companies could well afford to sell their terminals to the cities, providing they were guaranteed no loss of the business they had built up at a great cost; and it would be very hard indeed to estimate the relative value of a strategic location in a port, as compared to one not so favorably located.

In regard to railroad piers, the speaker has found from experience that they are seldom filled. In fact, in a publicly owned or operated port, ship schedules can be arranged so that those piers will be working to capacity. The speaker has known a time when the railroad piers would not average one vessel per week. That does not pay, and it is necessary to haul a great many tons over the railroad to make up for this terminal loss.

The Baltimore and Ohio Railroad Company contemplated, and made plans for, a pier and warehouse development at Tompkinsville, N. Y. It was not built, however, at the time it was being considered, because the financial market was not favorable. The Company then learned that the city might take the site and one thing and another led to the postponement of the development, and now the city has taken the site. This is only one case, but, in general, it shows why the railroads have not built modern terminals and warehouses at the ports to provide for future business.

Recently, a railroad man said to the speaker, "We have to build more terminals; it is costing us $1 000 on per diem charges". In other words, the cars from various sources are shipped to this road, because this company has arrangements with other roads not reaching the seaport, to handle their cars, and so many carloads were held on consignment in its yards that the per diem rate which had to be paid to the other roads amounted to $1 000.

If the cities or the public owned the terminals, they would be adequate, the railroads would not have the excess terminal charge, and, with adequate terminal warehouses, they would not have to provide so many cars.

In going through his files, the speaker found a reference to the comparative cost of building cars as against building warehouses. For example, take the Locust Point Yard of the Baltimore and Ohio Railroad Company, which has a capacity of 2 500 cars. There has not been a day in the speaker's experience with that railroad, in which that yard was not, as railroad men term it, "chock-a-block". For $1 600 000 a warehouse, including aisleways, mechanical equipment, etc., could be built to house the cubic contents of those 2 500 cars, so that it would work in with the yard and piers. The cars and land are worth $5 000 000.

If each of ten railroads was to build two of these warehouses, it would be equivalent to building 50 000 cars, as it would release that many cars now tied up in yards due to lack of storage room. Without property or tracks, the cars would cost approximately $100 000 000, while the warehouses could be built for $32 000 000.

The railroad companies are discriminating against the domestic as well as against the export shipper, and have done so for years past, in that they allow their cars to be used as warehouses, and that is the reason the warehouses are not built. It is easier for a shipper to ship on a through bill of lading and get that 15 days' free time, with the hope that a vessel will be available and that satisfactory shipping rates can be arranged before it expires, than to pay handling charges in the warehouse. The value per car and space in a yard is calculated to be $1 500. Interest, maintenance, and depreciation charges on the car and tracks are estimated at $172, while if the same carload is placed in a warehouse on the same basis of interest, maintenance, and depreciation charges, it will cost only $61.

Assuming that the average load per car is 25 tons, the annual revenue, on a storage basis, is $115, or a loss of $57 per car per year, if merchandise is stored in the car, and a profit of $54 per carload, if it is placed in the warehouse.

Since the Locust Point Yard is reasonably full of cars at all times, the speaker has used the yard capacity on a yearly basis for estimating. However, all material would not be stored a full year, and after deducting the handling cost, the revenue would be nearer $45, or $112 500 net profit, for the warehouse, against a loss of $142 000 per year, if stored in cars. These figures are general, but they are relative. The earning value of the car in service is not taken into consideration; the comparison is based on its use as a storage warehouse.

The value of the increased capacity and the removal of the necessity on the part of the railroad companies of buying high priced property in order to increase the capacity of their yards, with the likelihood of adding to congestion, are items which count for more than the actual earnings figured on a tonnage basis; and by keeping the cars more actively in use, the miles per car per day will be materially increased. This, of course, would mean greatly increased revenue, and, after all, the public pays the bill, and if the railroad companies could be relieved of excessive terminal charges, they would be in better shape to accept a reduction in freight rates, in which the public is so much interested at this time.

Take New York City as a concrete example, and assume that it will have its usual normal growth for the next ten years. Can any one imagine what it will mean to enlarge the yards and the cost of additional cars necessary to take care of the additional business? Now is the time to start building warehouses in order to prevent this necessary and expensive expansion.

Relative to the matter of warehouses, about three months ago, some interested parties had an opportunity to get a large manufacturing concern to locate at Baltimore, Md. It was practically all arranged, when, in figuring its operating cost to the *n*th power, the company found that Baltimore did not have proper warehouses for certain vessels, or rather certain ship lines, which were handling the raw materials that it wanted from Europe. This

meant that it would be necessary to ship those raw materials from New York to Baltimore to be manufactured, and have them shipped back again for distribution. Therefore, Baltimore lost that enterprise, and New York got it, because the latter city had a greater percentage of warehouses of that character, and also because it had the shipping lines carrying the raw material. This example is a point in favor of the better terminal, and is the strongest argument that can be brought home to engineers on this subject. There should be adequate terminal warehouses built at every seaport.

FUNCTION OF PORT TERMINALS AS CLEARING AGENCIES

By J. Rowland Bibbins,* Esq.

The United States Chamber of Commerce is deeply interested in, and is organizing to study more intensively and on a National scale, the transportation problems of the country as a whole. The speaker, therefore, will confine himself entirely to one of the broader aspects, as he conceives them.

Four distinct aspects of the subject have been outlined for discussion: First, the technical design, which relates to adequacy, capacity, and efficiency in technical problems, and which any experienced engineer can work out, given the problem of equalizing the capacity of main lines, terminals, and ships, such as will be necessary to handle the business of a port. The second aspect is that of administration, covering executive policy, service charges, co-operation with other transport agencies, development plans, etc. The third aspect is the merchant organization, a term happily devised by Dr. McElwee, covering such functions as banking, warehousing, forwarding agencies, etc., all relating to the merchant business of the port. Fourth, and most important, is the aspect of the port terminal as a gateway—the gateway of the interior. It is the funnel mouth, the clearing house for the nation's business with foreign lands, too often complicated by local business with which it has no relation. The gateway controls the production and the economic development of the interior in a manner often lost sight of in discussions of technicalities.

Commerce is interested in developing economical gateways and keeping them open and unobstructed. Commerce regards transportation as a unit element in the cost of doing business, that is, of distribution. Commerce pays the entire transport bill and does not single out any particular element, such as the port terminal charge, on which to concentrate its demands, unless conditions arise which retard, obstruct, and render the transaction of business unnecessarily costly or hazardous. The thought which the speaker wishes to convey is that in this discussion the total cost of transportation must be kept in mind, from the origin to the final destination of shipments. This may involve five or ten re-handlings, or even more for small lots, namely, from the producing interior, through the collecting agencies, the rails, through the terminal agencies and warehousing, the ocean carriers, through the foreign terminals, and distributing agencies; and *vice versa* on imports. This is the complex problem that confronts commerce, and, of course, the port terminal is an important element in that whole problem. After all, the essential criterion of all transportation is based on two things—Time and Cost, and these elements are involved in every branch of the transportation industry, representing an investment of approximately $50 000 000 000.

Back of these various coastal gateways is a great producing area of 3 000 000 sq. miles (with an additional area of 1 000 000 sq. miles in Canada which is closely associated with that of the United States). It is said that only 5% of the total business of the United States is with foreign countries, but the speaker suspects that 25% of the raw products flow through the ports.

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This internal production from farm, mine, and factory has grown up through long years of development and had adjusted itself to the pre-war plan of rates, routes, terminals, business organizations, and manufacturing locations (such as Gary, Ind.). The war dislocated the whole economic plan, that is, the relationships between location of producing centers, transportation costs, and distribution costs. Years are required for an industrial system to readjust itself to essentially changed conditions. If this dislocation continues, the future may force industry to gravitate to the seaboard in order to meet foreign competition. The truth of this statement appears in the great demand for a reduction in export freight rates from the far interior; the demand does not come from seaboard points. Therefore, unless the gateways—and by this the speaker means not only the water gateways, but the internal rail gateways as well—become so highly organized as to overcome this handicap of long and expensive interior transportation, a complete realignment of industry in the United States may be witnessed.

Commerce gravitates along the line of least economic resistance; in fact, if one can conceive of commerce and tonnage as water running down hill, one can visualize the Economic Divide—a hypothetical mountain range running diagonally across the country, determining which way that water-borne tonnage should naturally flow to foreign ports, that is, whether it should flow east or south, or to the west. It is not difficult to equalize the water and rail rates from a given producing interior region to a given foreign destination to locate definitely this Economic Divide.* Temporarily, the Divide may be diverted by sentiment or precedent; that is, if logic and the true cost of service dictate a certain route for internal tonnage to its foreign destination, and if it is found that the tonnage actually flows by an entirely different route, this may be the result of historical precedent, regional sentiment, preponderant rail development, or shipping policy. Eventually, however, trafficways for this interior production, in order to become permanent, must find and rest on their economic justification. Likewise, the gateways, in order to enjoy permanency and to meet growth, must retain their efficiency; and the gateways are the keys to the situation.

How shall this demand for continued expansion be met? Analyze the general business indices† of typical cities of the United States and it will be found that the general business which concerns transportation doubles in from 8 to 13 years. Cleveland, Ohio, is an example of a progressive community doubling its business in 8 years.

Now, even a modest port plan would require at least 5 years to organize and start *de novo*. It might easily take 10 years or more to complete the major elements, yet the ton-mileage of the United States doubled in 13 years, just prior to the World War, and its tonnage has increased as fast as the fourth power of the population. Here, then, is some measure of the problem

* See the speaker's paper, "Economic Lines of Gravitation for Overseas Movement", Am. Assoc. of Port Authorities, 1920; *Engineering News-Record*, 1919.

† Such as imports and exports, railroad tonnage, shipping tonnage, warehousing, bank clearings, post-office receipts, industrial output, auto-registration, electric railway traffic, school attendance, telephone traffic, etc.

of growth which concerns every seaboard municipality—every port terminal—and the widespread lack of appreciation of this rate of growth is responsible to a great extent for the lack of development. In other words, a different conception of port-terminal development is necessary, if Americans are to meet on a National scale the obligations imposed by an expanding commerce.

Let us now examine a typical large port terminal.* This port includes half a dozen competing railroad-terminal services, all interlocking as a result of previous development through 20 or 30 years of intense competition. Some of the terminals must actually be losing money in the effort, but they must keep going to secure their share of the traffic. This port city has grown solidly around the railroad terminals and the water-front district, enclosing them and restricting the possibility of their proper development.

Of the total car movement, only one-sixth is through land interchange, as compared with more than one-half in Chicago, that is, traffic having no business in the city proper. More than two-thirds of the total car traffic is interchanged between the roads, and two-thirds of this is done on the congested water-front, while only one-fourth of that car movement is direct export and import movement. In other words, city business and marine business is hopelessly interlocked in the attempt to carry on both in the same location. This emphasizes an important premise—that the water-front is not the place for city business, nor is it the place for rail clearing or storage operations.

Now, the peak load of this port is quite severe. The terminal handles 25% more cars loaded and empty during the peak load than during the normal month. It handles 50% more cars received immediately after harvest time, and, at times, the grain receipts are several times larger than the average for the month. The result is that cars pile up in the terminals by the thousands. The seasonal excess (over minimum) may be as much as 15% of the total car receipts, and months are required to eliminate this excess car holding.

This emphasizes the fact that the clearing capacity of terminals depends much on their mobile reservoir capacity—the "liquid assets" of the operating man. That point has indeed been emphasized in this discussion, but it means capacity of warehouses as well as of railroads. The railroads should not be required to provide all the warehouse capacity—a bad practice which greatly decreases the effective utility of railroad facilities.

Railroading is a continuous function; shipping is an intermittent one. The difference in the turn-around time in boats and cars must be met by reservoir capacity. Who is going to provide that capacity? That is one of the great problems of the port plan. If it is not provided, every harvest time will result in a vast car movement to the seaboard, the clogging of terminals, and a serious shortage of merchandise cars in the interior. The coal problem is similar. One 10 000-ton cargo would require 3 miles of cars standing, 2 000 or 3 000 truck loads, or 8 miles of trucks in line. Imagine the activity required in a port terminal designed to handle 20 cargo ships at a time, with an average of 5 days turn-around (which should be possible

* Based on a detailed technical survey of origin destination.

instead of 10, 15, or 20 days' turning time). What is needed at present is more movement, not more cars or ships. It is the cost of idleness that eats up the profits.

Summarizing this very brief discussion, commerce is interested in:

First.—The clearing capacity of the gateways during heavy traffic seasons. This is analogous to the problem of designing a power plant to carry a peak load.

Second.—The over-all cost of transportation from the producer to the consumer, including the important terminal cost. It is the total cost from arriving train to departing ship that controls.

Third.—The time consumed in transit, avoiding expensive and embarrassing delays.

Fourth.—Economic routes overseas which provide least time and least cost.

Fifth.—The provision for continuous development to meet future growth of traffic. In this respect, the newer ports—the younger ports—have a great advantage, because they are not handicapped by expensive precedent.

Sixth.—More consideration of the transport machinery back of the bulkhead, that is, rail, belt lines, and motor transport auxiliaries, are required in the port, as well as the water-side facilities and equipment, likewise so important.

Seventh.—The separation of city and water-side freight transport facilities is necessary to permit each to function without interference. This should be a major feature of the city plan.

Eighth.—Acceleration of carrier equipment. This may determine the success or failure of a port.

Permanency or supremacy cannot be guaranteed to any gateway, for, ultimately, Commerce will adjust itself and select its own trade routes and outlets, based on true economic stability.

LACK OF CO-ORDINATION IN DESIGN OF AMERICAN PORTS

By John Meigs,* M. Am. Soc. C. E.

Recently, a conspicuous official of one of the major ports of the United States, in an unfortunate moment of mental aberration, charged the Engineering Profession with responsibility for the lamentable fact that "this country to-day is secondary in importance to Great Britain and other European nations as a maritime power."

This ingenuous and surprising allegation was followed by the further count in the damning indictment that these hyper-technical malefactors—that is, engineers—or, as he neatly expressed it, "impractical and technical men," were animated by motives "sometimes prompted by selfishness, frequently by favoritism, and often by inexperience."

These initial strictures were then amplified by further charges of high professional crimes and misdemeanors on the part of various specific members of this Society, who had the temerity to design harbor structures not in accordance with his own secretarial views—this having been his vocation prior to his present incumbency of a responsible executive post.

If this highly placed official had additionally charged the impractical theorists of this Profession with the responsibility for the World War, for the present financial depression, and the unusual prevalence of mosquitoes on the Jersey coast, the sweeping indictment would have been complete, his animadversions would have been quite as just and logical, and no doubt it would have relieved his mind of the profound depression in which it was then apparently inextricably bogged.

Of course, such unmitigated absurdities as these would warrant no attention, save that they came from an incumbent of a prominent municipal-political office, and that they indirectly "point a moral and adorn a tale."

If, however, engineers reject this kindly meant solution of the problem of "what ails our American ports?", in fairness they presumably must suggest a better one. The uncontrovertible fact is, not that engineers have by their iniquitous dominance been responsible for the comparative backwardness of American port cities, but, on the contrary, that they have had far too little to do with their development, having been in most instances, until very recent years, notable in important port councils mainly by their absence.

It is true that the services of engineers have long been utilized in port design along the more technical lines of harbor planning and in the detailing of various port structures, but, in the larger aspects of the art, the general design and co-ordination of port layouts, they, unfortunately, have not been able to make their influence felt as they should.

Too frequent changes of political administration in the greater number of the ports have made impossible the adoption and carrying out of logical and comprehensive plans, however good the intentions of their proponents may have been, have rendered futile the well meant efforts of engineer subordinates to institute well considered programmes of development, and have resulted

* Cons. Engr., Philadelphia, Pa.

in piecemeal and inconsistent efforts at isolated betterments. Each successive, short-lived administration has entered into office as a rule suspicious of the plans and procedure of its predecessor, anxious to make an ornate showing for itself, and regardless of the troubles left by it in the hands of its successor.

To illustrate this precession of American port luminaries, the speaker may mention the fact that nearly ten years ago, when the American Association of Port Authorities was first organized, as a humble member thereof he became acquainted with practically all its membership, including the representatives of the ports of the United States and Canada. During this decade, the speaker has seen in the case of the ports of the United States at least three changes of administrative officers and, in some instances, four or five; while in the case of Canadian ports they have remained for the most part in the hands of the same executives for this entire period—men trained, salted, and seasoned by long and valuable practical experience. Not one of the municipal port officials of the United States then in office is now connected with port administration.

This applies, it is true, mainly to the chief executives of these various departments, but it affects also the working personnel to a considerable extent and makes impossible the proper carrying out of consecutive programmes of improvement.

The better balanced developments exhibited to Americans by European and by some Canadian cities—notably, Montreal—and the superior economies of operation possible thereby, are due in large measure to their rational conception of the proper function of a port administration as a genuine business organization rather than a political or social diversion.

When referring to comprehensive port developments, the speaker does not necessarily mean the somewhat flamboyant and spread-eagle projects designed to provide for the hypothetical needs of unborn generations of men and unbuilt navies of ships with which the port fathers have amused themselves in so many recent instances, admirable in themselves though these plans have often been. Rather, he more especially wishes to emphasize the necessity for proper balance in port layouts, be they large or small in size, and an economic co-ordination of the various parts and functions of the port.

Provisions for the receipt and transfer of freight to and from land and marine carriers are essential, of course, but of equal importance must be considered the provision of facilities for its warehousing and merchandising. The complete port consists not only of its channels, anchorages, piers, quays, and transit sheds, but also of proper rail and road transportation facilities, and, not least, ample provisions, both outdoors and under cover, for the short and long-time assembly of cargo and the protracted storage of it in warehouses for eventual reshipment, for resale at favorable opportunities, for repacking, assembling, manufacturing, or other preparation of it for the market.

Have American municipalities devoted the proper study to this important subject as a scientific problem, or are the ports being permitted just to grow, "Topsylike", along the lines of least resistance? To any one familiar with the

trend of port development in the United States, it is not necessary to answer this question. He will know only too well that we are merely drifting.

Much money is being spent in the improvement of the harbors—tremendous expenditures in the aggregate are being made—but how much of this spending is wise and how much of it unwise, or even profligate, is a serious question. When two ports of approximately equal commercial movement are found, one with nearly twice the wharf space of the other and still feverishly engaged in spending on more, and more, and more wharves, it looks as if satisfactory explanations of the "whyness" of this spending would interest the citizen taxpayers who are providing the funds for the possibly misguided expenditures.

When port officials devote their energies solely to constructing spectacular new piers, and entirely neglect the equally important railroad and warehousing facilities to back them up efficiently, and the proper improved mechanical equipment to facilitate the despatch of cargo, it is time for a halt until the entire system of port administration can be placed on a genuine business basis.

Permanent boards, composed of business executives, operating men, and trained engineers, should be placed in charge of these matters, paid decent compensations, and permitted and required to stay on the job. Then and then only—when the ports are administered as business enterprises and not as political opportunities—can Americans hope to compete in operating efficiency with well organized overseas competitors.

No such reckless and extravagant use of water frontage is observable anywhere in the world as in the harbors of the United States, which, provided by Nature with facilities almost ideal as compared with the great ports of Northern Europe, have been developed in truly frontier fashion, sprawled out over many times the length and area of other world ports of corresponding commercial importance, and operated with consequent inefficiency.

Per foot of steamer berthing space, American ports show an annual unloading capacity of from one-third to as little as one-tenth that of well planned and properly administered foreign harbors. When one considers what this means in decreased costs of every kind—both in connection with construction and operation—the greater administrative facility, and the more efficient management in every particular, the facts in the case demand most serious attention.

As a typical American example, the municipality of Philadelphia, Pa., has about 15 000 ft. of highly improved, deep-water, wharf frontage or berthing space publicly owned, 65 000 ft. of additional deep-water berths under private control, and, approximately, 50 000 ft. of additional berthing space suitable for coastwise and river-trade purposes, a total of nearly 130 000 ft. of improved wharf frontage. Not a square foot of warehouse space in the port is municipally controlled, and, compared with foreign ports, there is relatively little under private ownership. The warehousing is done largely on the pier sheds, where it was never intended to be—and should not be—permitted; and no effort whatever is made to utilize the port equipment intensively.

Abroad, the Port of Manchester, England, handling but slightly less foreign business than Philadelphia, possesses only 34 000 lin. ft. of berthing space, less than one-half of the deep-water berthage in Philadelphia and about one-quarter of the latter's total berthage; but, on the other hand, Manchester has warehousing accommodations amounting to approximately 3 200 000 sq. ft. as against Philadelphia's inadequate quota of 1 200 000 sq. ft. In addition, the Manchester docks are equipped with most elaborate mechanical handling equipment designed to facilitate cargo loading and unloading and to make every linear foot of frontage of maximum cargo capacity.

A little nearer home, in Canada, the Port of Montreal manages somehow to handle an annual foreign tonnage of nearly the size and importance of Philadelphia's deep-water berthing accommodations—and accomplishes this in an open navigation season of less than 8 months, or only two-thirds of Philadelphia's working year.

The case of Philadelphia is not mentioned as unique. It is merely typical of current American practice along these lines, and nearly all the other ports of the United States are justly subject to the same criticism.

Are these inconsistencies of performance to be accounted for by any radical differences in classes of cargo handled, or the industrial surroundings of these several ports? Most probably they are not; and, also, most certainly the current practice on one side of the line or the other is wrong. Are the ports of Manchester and Montreal handling more tonnage per linear foot of available berthing space than they properly should and, at the same time, show a decent and kindly regard for American sensibilities; or, possibly, has Philadelphia and her sister ports more piers and berths than they have any legitimate need for, and which they have not yet learned to use properly?

If, however, the municipal authorities have shown lack of forethought in the study of these matters, private terminal managements cannot be accused of like carelessness, as some excellent examples of well balanced design are exhibited in their plants.

In the Bush Terminal, in Brooklyn, N. Y., for instance, with a total berthing space of approximately 16 000 lin. ft. and a pier area of more than 1 000 000 sq. ft., the warehouse area is in excess of 2 000 000 sq. ft. This terminal presents an exceedingly interesting study, in that although it is by no means a model from a construction point of view, it may be assumed to be an almost ideal layout from the standpoint of the balancing and proportioning of its various facilities in their relation to each other and to the general plan.

The present layout being the result of a gradual growth of several decades, from an insignificant beginning, its various piers, warehouses, railroad, and other facilities have been constructed one unit at a time during successive years in response to the actual demands of the business of the terminal, and, in its present state of completion, it may be fairly considered to be planned almost exactly to meet the accurately determined needs of this particular location. This great commercial terminal is commended to city fathers for their careful study.

Decided progress was made during the World War in scientific designing in the case of some, but not all, of the Government terminals constructed for

the shipment of munitions, and these steps in advance might well be emulated by municipal port planners.

In the great South Brooklyn Army Supply Base, a most radical departure in terminal proportioning was made when, in order to balance 7 800 lin. ft. of berthing space and 585 000 sq. ft. of pier area, there was provided more than 4 500 000 sq. ft. of warehouse floor area. This proportion of warehouse space to pier area of more than 7 to 1 may be far in excess of the correct balancing of these facilities in the average commercial terminal. This, however, is a preferable fault to the contrary one of insufficient warehouse space and a superfluity of piers.

It is poor business to build piers at from $5 to $10 per sq. ft. of deck area, and permit them to be used as storage warehouses, while the latter class of structures can be provided at from one-quarter to one-half the cost of pier area.

When comparatively cheap warehouses and inexpensive mechanical handling equipment will enable the expeditious clearing of the decks of expensive piers and permit of doubling, or trebling, the number of vessels capable of being accommodated at their berths, there would seem to be considerable virtue in a policy of port development calling for liberal warehousing provisions and the installation of ample cargo-handling machinery.

Is it not time that American municipal port executives ceased specializing in piers, and piers alone, and commenced an active campaign of providing other port facilities in proper relationship to their wharves?

Most of the seaboard cities of the United States need improved intra-harbor rail facilities, effective belt-line railroads, up-to-date mechanical equipment, and extensive increases in warehousing space incomparably more than a far-flung series of individual piers practically unrelated to each other and the general port plan.

It has been argued that railroad and warehouse installations are not proper objects for the expenditure of public moneys, that it is all right to build piers, but all wrong to provide the co-ordinate facilities which will make the piers of maximum use to the community. This is the most egregious stupidity.

Who can say where the proper functions of the municipality stop and those of private capital begin? The propriety of such expenditures can be determined only by the extent and urgency of the existing needs of the community, by the exigencies of each actual case.

The speaker is no advocate of undue paternalism in government, and he thoroughly believes that private capital should be encouraged and assisted in all legitimate lines of investment. Private management is nearly always superior to that by governments—be they municipal, State, or Federal—in point of economy, efficiency, and almost every vital factor of operation. If private money will provide all the port facilities, docks, railroads, warehouses, and what not, let it do it by all means—under proper public supervision of the general plan of these expenditures—but whenever private capital fails to take advantage of its opportunities to be of public service, then the public itself must take hold and provide rationally for its own needs.

In his discussion, Mr. Long* has already given some illuminating data on the subject of warehousing principles, and the figures previously quoted are merely random examples selected to suggest the possibilities in this line of investigation. The subject is certainly worthy of more careful study than has yet been given it, and the results in the way of improved port economics to be reasonably expected from a careful and conscientious investigation of the entire subject, and a practical application of the vital facts thus discovered, would amply justify its costs.

Let the municipal authorities wake up; let them show a little ordinary business sense; let them give their engineers and shipping experts a chance to work out their port problems along scientific lines—that is, in accordance with the dictates of sublimated common sense—and the United States will no longer remain as the before-quoted distinguished critic has stated, of "secondary importance to Great Britain and other European countries in maritime trade", at least, in so far as the proper economical and expeditious handling of maritime commerce at the water-gates is concerned.

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* See p. 164.

A BRIEF COMPARISON OF AMERICAN AND FOREIGN SEAPORTS

By W. Watters Pagon,* M. Am. Soc. C. E.

To persons content with superficial observation, the most striking fact about the two groups of ports—European and American—is their marked dissimilarity. Yet when a more intimate study is made, one finds, of course, that much the same factors are operative, and the differences that have developed on the opposite shores of the Atlantic are no more marked in their real essence than the differences in manners and language between the British Isles and America.

The fundamental difference is largely topographical. The speaker cannot recall an American seaport of consequence where he was not asked the question, "Do you not think that this is a wonderful natural harbor?" and, doubtless, it was, for most American seaports are natural harbors. A port, however, is more than a harbor, and depends for existence and prosperity on commercial and political geography and on the business acumen of its traders. Because of this many of the European ports are located on rivers, far from the coast, and their harbors have largely been dug, piecemeal, from the land as additional space was needed. It would be quite accurate to say that European ports have natural wharves and artificial basins, whereas American ports have artificial wharves and natural basins. The obvious consequence of this—somewhat accentuated in America by the individualism of the pioneer and the temporary character of his construction—has been to foster the building of docks in Europe and of piers in America.

On first thought this distinction is all important, yet the speaker cannot but feel that it is of little importance. A quay wharf is nothing more nor less than half a pier, and whether the wharf is a masonry bulkhead retaining an earth-fill, or a piled construction, can only affect the economy of doing business, through its influence on capital and maintenance charges. In fact, though Liverpool is typically European in its materials of construction, it is quite similar to American ports in its layout, and other foreign ports have layouts containing piers—even piers of American pile construction.

Just why tideless docks have been used so extensively in all ports of Europe in recent years is not entirely clear. In Antwerp, for instance, the earlier construction was on the principle of the marginal wharf, very much like that at New Orleans, La. In London, there is a theory that the earlier basins were built quite largely because they provided a water area that could be fenced in to protect shipping from pilferage; which theory is supported by the inscription on the West India Docks, dated 1802, namely, "* * * this Range of Buildings Constructed together with the Adjacent Docks, * * * for the Distinct Purpose of Complete Security and Ample Accommodation (Hitherto not Afforded) to the Shipping and Produce of the West Indies at this Port * * *". Hamburg and Copenhagen have used the idea of docks, without gates, because it lends itself to the requirements of their "free

* Cons. Engr., Baltimore, Md.

ports". Rotterdam has real need for large basins—even if no quays were provided—because a large volume of its business consists in transshipment from Rhine and canal barges directly to vessels that are moored to pile dolphins and buoys in the docks. Yet, of course, there are many advantages to be gained by tideless docks in such a port as London, where the tidal range is 25 ft. and the tidal currents are strong.

Therefore, it cannot be said that the common American type is better in general than the European type, or that it should always be followed in the United States. The better principle to lay down is that both types have their advantages, and American ports should develop their newer port works along either line, according as the local requirements favor the one or the other. To follow the American plan exclusively is to blind oneself to the possible economies of the other, a fact which has been appreciated by some of the port authorities, and certainly it is contrary to the ideal of the terminal engineer, which is to build so as to gain for the owner the maximum ultimate economy.

For the purpose, therefore, of understanding the other factors that influence the type of development, let us set aside the factor of tidal range—a fundamental one to be sure, yet one that is purely local to every port.

Next in order of discussion is, then, the matter of relative permanence of construction. On this subject many elements have a bearing, for example, relative age of ports, tides, and other physical elements, seat of ownership, density of business, etc.

Relative age of ports will be discussed more fully subsequently, but those foreign ports the age of which is so great that their business activities are well established, can well afford more permanent structures than the recent ports in this country. Yet, this consideration did not cause Manchester to build cheaply, although its port is an infant, even when compared with the Atlantic ports of the United States, and when Antwerp was opened to trade again by Napoleon, the structures built were of permanent type. Railroad-owned Southampton is also an exception; so probably this reason does not generally obtain.

Such influences as tides may have a considerable bearing on construction where structures are in the tidal range, but they would have no bearing within the docks where the water level is almost constant. In the United States the range of tide is not a determining factor, because the American type of piers may be found at Baltimore, Md., with a tidal range of 1 ft., and at Boston, Mass., with a range of 13 ft.

The questions of seat of ownership and density of traffic are closely related. Of course, the tendency in all countries is to build public works of permanent materials, and this may have considerable bearing in those ports where city or State or other public authority controls the works. Manchester and Southampton, however, are privately owned, and here permanent construction is also found; and the privately operated "free ports" are of similar nature. Even in ports which follow the leasing system, where the lessee's wishes govern largely, there is no marked difference. Therefore, it is probable that the mere fact of ownership has no direct bearing, to the exclusion of other factors.

On the other hand, delegation of ownership or control of a preponderant portion of a port to a single body—whether public, semi-public, or private—is conducive to concentration of the business over the minimum of space. Density of traffic also goes hand in hand with permanence of construction. Thus, a line of "jitneys" may amply provide for light, suburban, street traffic, but subways are the only solution for heavy metropolitan service, and heavy capital expenditure may be economically justified under the latter conditions.

This is, the speaker believes, the most important factor, and a little detailed study will emphasize the point. It has been stated that certain European ports handle ten times as great a tonnage per linear foot of quay as certain American ports. Assuming that cost of construction of berthing space alone in the two countries is in the ratio of 3 to 1, then it follows that those foreign ports are doing an amount of business certainly three times as great per dollar of capital investment. If only this much is true, it certainly does not speak well for the American extensive system when compared with their intensive system. There are also, however, many other correlated gains which follow from intensive development. There is a further saving in terminal investment and operating costs by reduction in railway trackage and facilities; a similar saving in warehouses and transit sheds; a saving through the changed conditions which justify labor-saving machinery by permitting its constant use; a saving in truck mileage to outlying terminals; a saving of water-front mileage and, therefore, a reduction in value, thus permitting more widespread use of the shores for industrial plants the materials of which are water-borne.

By comparison, the most potent element of weakness in the ports of the United States is the decentralization of a pioneer country. Such a condition as that found in one American port, where only 1% of the harbor frontage is owned by the public, would be impossible in Europe, where the ports are highly centralized, for administration as well as for physical layout. London has its Port Authority controlling nearly all the important quayage. In Copenhagen, 58% of the quayage is controlled by the Harbor Authority, and 70% if city and State are included. Liverpool and Manchester are almost entirely in the control of the local authority, and the ports of the Low Countries are similarly held.

New Orleans, La., and San Francisco, Cal., are on a parity with these ports, and they are indicative of the success of the idea in America. In San Francisco, the Authority has consistently striven to increase the density of traffic by yearly comparisons of the tonnage handled per square foot of pier. The space allocated to a shipper is contingent on the intensity of use shown previously. The Authority at New Orleans reserves the right to place vessels at wharves which are not in use, even though they are allocated on yearly leases.

In marked contrast with the natural facilities offered by American harbors are some of the development difficulties to be observed in European harbors. The extreme range of tide has been mentioned, with its attendant system of locks and dikes. Contrasting with this is the situation at Copenhagen where the tidal range is slight, but where the tidal currents are so strong that a dike 7 300 ft. long has been built across the Sound, with locks for vessels

and sluice-gates to control the currents. The newer portion of the harbor has been reclaimed from the water and enclosed by about a mile of breakwater. Amsterdam for more than a century had difficulty with the depth of water through the Zuider Zee, and has built two canals to the North Sea—one of which was not successful—to maintain its position. The later one is second only to the Panama Canal in section, and was built at a cost of about $25 000 000. Rotterdam, located more directly at the mouths of the Rhine, has had to face the same problem of maintaining its channel depth to the sea, and has expended only slightly less than its neighbor. Probably the most courageous of all is Manchester, which, aspiring to become a seaport, spent nearly $90 000 000 on a ship canal, and now is almost on a par with its former port—Liverpool. In some ports, the State has assisted the city, but the greater the proportion expended for aids to navigation the less there was available for quays, sheds, warehouses, etc., and, therefore, the greater the need for increased density of traffic.

One weighty difference between the continents lies in the fact that American tonnage is predominantly for export, whereas, in many European ports, it is for import or for transshipment. This is notably true of the grain trade. However, Amsterdam and London, for instance, have extensive warehouse facilities for goods awaiting re-export, which largely overshadow the other port facilities. Much of the wealth of these European countries has been derived from such storage of goods awaiting a favorable market, in addition to the profits accruing to the local ship-owners from transport of the cargo.

The "free port" is peculiar to Northern Europe, where it has been fruitful. Developed to meet a local situation, it has spread to foreign countries. From the traffic standpoint, it is much the same as an interurban railway terminal, where the dense mass of passengers from subway or elevated railway are distributed to the radiating lines which end there, or *vice versa*. From the economic standpoint, it is a wedge, a re-entrant salient, in the customs frontier of a country, which extends the commercial freedom of the seas to include a safe harbor where goods may be safely transferred, blended, manufactured, or stored, awaiting continuance of the journey to other countries. It is a "bay" connected with the free ocean and enclosed on three sides by the customs, whereas a bonded warehouse is an economic "island" surrounded on all sides by customs.

Because of its nature, a free port is, in general, owned by the State or Harbor Authority and leased to a private lessee. When the Government of the United States adopts the principle of the free port, which now seems entirely possible, it will have at New York, Baltimore, Norfolk, and other ports, a series of Ordnance General Supply Depots with berthage, railways, storage, fencing, and all other adjuncts of a free port at hand ready for lease. At least one of these ports can easily be developed to equal the present size of the free port at Copenhagen. For equal success, however, there must be aggressive commercial activity to provide the business.

In spite of superficial differences, therefore, there are few factors of importance which are not operative on both continents. Of course, most Euro-

pean ports have much the advantage in point of age, for Tacitus writing in A. D. 61 mentions London as having "a number of merchants and trading vessels", and several Continental ports antedate the Norman conquest. Yet the rise and fall of ports has been rapid, and whereas Antwerp had become one of the foremost ports of the world in the years from 1500 to 1560, she had declined to almost nothing at the end of the century, owing to civil wars and political dominance of other States. Thus, age can create only an historical background, which may facilitate the inauguration of some new venture, but lack of age has been no deterrent to new ports where the local spirit was aggressive and conditions were ripe.

Whether conditions are ripe for American seaports no one can predict. It hinges quite largely on the continuance of the merchant marine. No European city has become great through the activities of its non-citizens, but through those of its own people, who owned their ships and brought the goods to storage under their own house roof or warehouse. The profits from vessel and cargo went to support the Rubens, the Michael Angelos, the engineers, and other professional men, as they will in the United States—if the merchant marine is made to flourish—to those ports the citizens of which own and operate their vessels and lines.

SOME OBSERVATIONS ON PORT FINANCES

By Edwin J. Clapp,* Esq.

The speaker is not going to discuss the subject of an ideal method of collecting port revenues, nor consider, except briefly, the various expedients now practiced in raising enough money to operate port facilities. He is going to discuss, primarily, the practical financial difficulties that confront public port construction work in the North and South Atlantic outports, because of the established practice of the railroads in offering free berths to steamships in the foreign trade. All the Atlantic ports of the United States, except New York, are called outports.

Briefly stated, the situation is as follows: Each railroad at its chief port, often at several of them, maintains a complete set of oversea terminals, consisting of water-front yard with switching equipment, open and covered piers or wharves, grain elevators, cranes, and other machinery for handling bulk freight. Generally speaking, it maintains and operates this ocean terminal as part of the railroad system, just like the team tracks and freight houses of its land terminals. Rail rates cover not only the cost of transportation, but the often greater cost of maintaining and operating the terminals. No charge is levied on the motor truck which backs up to a freight house to deliver or receive freight. No charge is levied on a steamship that backs up to a pier to deliver or receive freight. The interest, depreciation, maintenance, and operating costs of the pier are covered by the rail revenues earned on goods hauled to and from the ship.

The outports, intent on increasing the volume of traffic flowing through them and the number and frequency of their overseas services, have found marked disadvantages in this system of railroad ownership of pier facilities. The railroad, having invested heavily in a terminal, tries to monopolize all competitive traffic that passes over it; that is, all traffic to and from points reached by the railroad pier owner or its connections—and this includes practically all the traffic the steamship carries except local port business. Other railroads are kept from interchanging traffic with the steamers when the pier owner refuses to switch their cars, or levies on these cars such a heavy switching charge that the other roads are discouraged from carrying traffic the revenue of which is thus diminished. In either case, the steamship line tends to find itself confined to the service of the carrying power and the soliciting force of one railroad, instead of all the lines centering at the port.

Likewise, the railroad pier owner generally bars or restricts the use of its pier for the accumulation of local cargo delivered by teams, lighters, or the short-distance motor-truck common carriers. This is business on which the railroad gets no revenue. Such traffic occupies pier space which railroad revenue freight might use. It is good business for the railroad, but bad business for the ports the traffic of which would expand if all piers were freely open to all inland carriers by rail, highway, or water. The railroad, however,

* New York City.

handling all of 500 000 tons of freight moved through its ocean terminal, can make more money than by handling one-half of 800 000 tons. At least, so the railroad officials have calculated.

An efficient port is a funnel through which are poured the exports and imports of a wide hinterland. Railroad ownership of ocean terminals has interfered with the freedom of that flow in the outports. The outports see their disadvantage when they compare themselves with publicly owned ports like Montreal, Que., Canada, and New Orleans, La., where the berths for steamers are owned by the Dock Board and the facilities are connected with each railroad by a public belt line. Each steamer is thus cut off from exclusive connection with any rail carriers. For a moderate, uniform switching charge, covering only operating cost, the belt line switches cars between any berth and any carrier. Every steamship line has every rail carrier working to create traffic for it, and all on equal terms. In New York, steamers do not dock at the railroad piers, but across the harbor at piers in Manhattan, Brooklyn, or Staten Island. Railroads deliver freight to the steamers by lighterage. The intervening water serves as a belt line to cut all the water carriers from all the land carriers.

An outport sees that public piers are preferable to railroad piers from a traffic standpoint. Moreover, the outport sees the publicly owned ports going ahead with great extensions, such as have been completed or are in process at New York, Montreal, and New Orleans. In general, it can be said that in the provision of new facilities, the outports have stood still; the railroads are building no new piers and have built none for years. This is partly because the export and import business has declined from a major to a minor element in the traffic on the railroads, and the railroads have not the money to build to meet the less important needs. The insistent demand for money to rehabilitate the road and equipment of carriers, and to expand local terminals for domestic traffic, will absorb for years all the funds the railroads can raise, and there will be nothing left to spend on ocean terminals.

Thus, not only is it better policy for the city or the State to supply piers for oversea carriers, but it is the only way in which port development can proceed. However, if an outport erects a pier, or a group of piers, it finds it difficult to make the steamships pay anything for their use, because a few blocks away the railroads offer their piers rent free. The railroads recoup themselves by their rail earnings. The city has no direct earnings out of which to pay the cost of carrying and supplying piers free to steamship users. Few cities are financially able to carry this annual loss for the sake of the general advantages which port development brings to commerce and industry. How are the outports to finance their new public piers, meet the cost of their interest, maintenance, depreciation and supervision, or operation? There are four ways in which this money can be raised.

First.—The piers can be supplied free and the annual cost taken out of the city taxpayers. This is not practicable. Indeed, it has become customary for the voters or their representatives to authorize port expenditures only on condition that they are to be self-supporting.

Second.—The annual cost of a pier can be met by a dockage charge, levied against the ship each time it uses the pier and proportioned to the size of the

vessel and the length of its stay. This is a main source of revenue at New Orleans, where it is supplemented by a "sheddage" charge levied on the ship for the use of covered space, and by an annual "preferential assignment" charge per square foot of space paid by the steamship line for a semblance of permanency in its berth. In some other ports, these charges against the ship are commuted into an annual rental, as at New York, where the pier is turned over to the steamship line for its exclusive use.

Third.—The annual cost of a pier may be met by the imposition of a "wharfage" charge levied on the goods that pass over it. These revenues may be supplemented by storage charges levied on the goods which do not simply pass over the pier, but remain there for a time. This method of financing piers, developed by the railroads, is practiced at the South Atlantic ports. They have long collected charges according to a "wharfage, storage and handling" tariff, the charges being added to the rail rate on southeastern export and import traffic. For years these charges have been far too low to meet the cost of maintaining and operating the piers. Since January, 1921, a new tariff has carried charges designed to be adequate.

At some ports such wharfage charges are not added to the rail rate, but paid by the railroads out of the rates. This is done at Montreal. Its chief source of revenue is wharfage, most of which is paid by the railroads out of their earnings. Montreal has a very scientific system of raising its revenues. The wharfage charges already described are kept high enough to provide interest on the piers. The berths are then leased to steamship companies at rentals sufficient to carry the annual cost of the sheds. Elevation and storage charges make the grain elevators of the Montreal Harbor Commission self-supporting. Switching charges paid out of rail rates to the harbor belt line are designed to make the belt line carry itself. At Galveston, Tex., a similar method prevails. The ocean terminal facilities are owned by the Galveston Wharf Company. Its revenues come mainly from wharfage on goods allowed by the railroads out of their rates, and dockage on ships, paid by vessels according to their size and duration of stay.

Fourth.—The entire cost of maintaining and even operating piers may be met out of railroad revenues, as is the case with the railroad-owned piers at the outports.

As it is not practicable to propose that publicly owned piers at the outports shall be maintained at the expense of taxpayers, the only way to finance them has been by the dockage or rental method and by the wharfage method.

To rent public piers on a self-supporting basis will long be a matter of great difficulty at the outports. The speaker is not unmindful of the experience of Philadelphia, Pa., a railroad port which, ten years ago, bravely "took the bull by the horns", built public piers, and rented them for what they would bring, alongside railroad piers offered free. For many years the rentals were almost imperceptible. To-day, some of the newer berths rent on a basis to carry them, but the net income of the Dock Department, after taking out administration and maintenance expenses, is an insignificant return on the $12 000 000 expended thus far. Philadelphia frankly set out to carry its piers, like its highways, at public expense. No other outport can afford to imitate her.

In the speaker's opinion, it would be a mistake for an outport suddenly to attempt to finance piers on the basis of rentals or dockage collected from ship lines, even if this method were practicable. Everything tends to gravitate to New York. Free berths at the outports have been an inducement to more than one steamship line to establish there. When the railroads were built from the seaboard into the Middle West, the Erie Canal had already concentrated the exportation of grain and grain products at New York. The new railroads terminating elsewhere set out to induce the flow of this traffic through their ports also. They offered the steamships free berths and they also offered larger earnings than obtain at New York—larger steamship earnings because of the "differential" rates (lower than those which then applied to New York), which the railroads terminating at the outports were willing to accept. The outports have fought the abolition of the differentials. They should go slow in abolishing free dockage. Examination of export and import figures proves that the drift to New York is not growing weaker, but stronger. Such new lines as the outports get are generally the result of small beginnings. A free berth is a constant inducement to such experiments.

Financing new public piers at the outports by means of wharfage levied on the goods presents no complications at the South Atlantic ports where the rail carriers already levy wharfage and handling and storage charges, in addition to the rail rates. When new public piers are built, the rail rate will set the car on the city pier. Then the city can levy the regular tariff, handling, wharfage, and storage charges. These are designed to be sufficient to carry the old railroad piers which suffer under very heavy maintenance costs. The tariff charges, therefore, should more than carry new modern city piers.

At the North Atlantic outports, financing public piers by wharfage charges will not be so simple, because these charges cannot be added to the rail rate. As explained, the rail rate to a North Atlantic port includes delivery alongside the ship. The railroad, without additional charge, unloads the car and charges no wharfage on its contents. At a city pier, the railroad would deliver and unload the car. If the city tried to charge wharfage on the contents, this charge, added to the rail rate, would throw the cost of shipping *via* this pier "out of line", compared with the adjacent railroad piers. That is, at North Atlantic ports, if wharfage is to be collected at city piers, it must be collected not from the shipper in addition to the rail rate, but from the railroad out of the rail rate. This would be no new practice for the railroads. Already out of their rail rates they pay a wharfage sufficient to cover the overhead and the operating cost of their piers; they pay this wharfage to their own terminal units. Why not pay it to a new independent terminal unit which relieves them of the terminal service? Then the railroad would have the same net revenue for its haul whether the shipment passed over its pier or the city pier; in either case, it would retain the rail rate less a terminal deduction for wharfage. Such rail-rate terminal allowances made to city terminals by the carriers would make city terminals self-supporting. This would make it simple to obtain public funds to any desirable amount.

In other words, this is the situation: In the North Atlantic outports, ocean terminal facilities have been supplied by the railroads the rail earnings of which include a quota to pay for the upkeep of these terminals. Expanding commerce, larger ships, the advance of the engineering art, all require new and improved facilities. The railroads are financially unable to supply them. They can be supplied out of public funds, if the railroads will make to the new terminals the same terminal allowance they would make to their old terminals. Of course, the carriers would be glad to have the outports build piers and make no terminal allowance at all. They make no terminal allowance to the city piers at Philadelphia. On traffic which the railroads carry for movement over these piers, they retain the full rail rate, both that part collected for the haul and also that part collected for the terminal service, which includes supplying a pier. The cost of supplying the pier is thus thrown on the city, which shifts part of it to the steamship lines as rentals, but shifts most of it on the taxpayers.

To reduce the suggestion to concrete form: Suppose a North Atlantic outport builds a modern terminal unit, consisting of open and covered piers, a grain elevator with galleries to each steamship berth, a supporting warehouse, an adequate railroad yard, and a belt line cutting the main break-up yard of each rail carrier. Each rail carrier would pay the belt line an adequate switching charge for taking a car and setting it at its berth in the new terminal. This would supplant the service rendered by the railroad's switching engine in moving the car from the break-up yard to its own pier. The railroad car at the public terminal would then be unloaded by the railroad's own men sent there or, better, by the terminal's men, and the railroad would pay a proper handling charge for the service thus performed for it. Finally, the railroad would allow the terminal a per ton wharfage equal to the interest and maintenance cost of its own piers distributed over the tonnage handled there.

The railroads themselves would profit from such an arrangement. They would be supplied with new and additional facilities without cost, save as they got traffic to move over those facilities, and then the cost would be the same as that now assumed in the case of traffic moved over their own piers. At some of the outports, the carriers stand enormous losses by carrying cars of exports under load, because they have no adequate pier space to hold them. The initial wharfage allowance required of the railroads could be reduced in the course of time. The city's overhead on its terminal would be less than that of the railroads; the city could get its money for 5%; the railroads pay 6 per cent. The railroads pay 3% taxes; there would be none on city property. The overhead on the money invested would thus be 4% per annum less than if the railroads themselves built additional terminals. Maintenance at the new piers would be far less than at the old railroad structures.

The wharfage required of the railroads, in the course of time, could be further reduced as new sources of revenue arose for the new terminal. The larger steamship lines seeking it could be charged a "preferential assignment" for regular berths. Some of the lines would require special types of shed or equipment, but such lines could pay interest on the special facilities provided for them, a rental carrying the pier superstructure, as at Montreal. Adequate

tolls would be collected on freight carried to and from the piers by trucks and lighters. In all likelihood, the wharfage required of the railroads would be in time only a fraction of what their own piers now cost them in wharfage. It is not impossible that in time the new terminal would become quite self-supporting. It is a plan by which the railroads could gradually work out of the heavy terminal expense they now carry on export and import traffic.

The speaker realizes how sketchy this discussion has been; he knows the difficulties involved. The new terminal should be built and operated only by the highest type of public commission such as have been described by Mr. Cowie and Gen. Beach. The best commission or best authority is one representing the business interests which handle that transshipment which is the port's function. On this Port Authority, both railroads and steamship companies should have adequate representation. Of course, the new terminal units must be built gradually; the railroads should not be asked to help finance terminals which will simply empty those already in existence. There is no use in bedeviling the railroads; but for them there would be no outports; they developed them. The time has come when further development is beyond their financial powers, when ideas of the benefits of competition no longer extend to approval of separate railroad-owned ocean terminals. The speaker believes that the railroads will be found ready for co-operation in the manner herein outlined. If they were not ready, he believes that the Interstate Commerce Act, the Transportation Act, and the Merchant Marine Act, give the Interstate Commerce Commission power to compel such co-operation. Pending construction of public terminals, the main evils of the present railroad-owned piers can be eradicated by the introduction of reciprocal switching among them, the switching charge to include adequate wharfage for use of one carrier's pier by another.

What is here proposed is that the transition from railroad to public ownership of ocean terminal facilities in the North Atlantic outports proceed without abolition of the present railroad practice of allowing out of rail rates a wharfage sufficient to provide interest and maintenance on piers for oversea carriers. It is proposed that a city commission of port business men construct a single terminal unit, as part of a port plan, and that the rail carriers allow to this new terminal, out of rail rates, the same wharfage they now allow their own terminals. It is asserted that this initial wharfage paid by the railroads can soon be reduced, because of lower overhead and maintenance costs applying to the public terminal and because the terminal will develop revenue from "preferential assignment" leases and outright berth rentals, as well as from wharfage on traffic brought by motor trucks and lighters, which will have free access to the new piers.

Neither the outport nor its railroads can afford to see its port facilities stagnate. An attempt has herein been made to present a plan whereby these facilities can be fairly developed, to the advantage of both railroads and the port, without disturbance of existing rate or traffic conditions and with that proper observance of the local situation which must modify any attempt to attain ideal conditions.

IMPROVEMENT AND DEVELOPMENT OF PORTS

By Carroll R. Thompson,* M. Am. Soc. C. E.

In the improvement and development of ports there is no question of more vital importance than the powers that are vested in the body entrusted with the control and administration of port matters. The best laid plans for increasing efficiency will fail in their ultimate purpose unless the port control is such that the various unit facilities can be co-ordinated into a smooth working whole. A single port unit may be designed and built and be capable of operating with the utmost economy and dispatch, but unless the other units are coupled with it to produce the same efficiency, the whole operation of the port will be inefficient. The absolute regulation of each and every element entering into port problems must be controlled by a port body, which, in turn, must be established, organized, and maintained so that it can be administered exclusively as a business proposition.

The work of the engineer is dependent on this control. How can the engineers of a port commission build supporting warehouses to operate in conjunction with pier sheds, unless the commission has the authority to build warehouses? How can the rail facilities in a port be operated to their utmost efficiency unless the belt line is operated on a belt-line principle? What is more discouraging to a port official than to find one of the port's most modern and fully equipped general cargo piers on which a preferential is given to a particular railroad company's freight, or on which one railroad is permitted to run its cars to the exclusion of all others? When such a condition exists there is only one result: The railroad freight that is not permitted to be run on a pier must be rehandled at a distant point and transported to the ship either by lighter or truck. The belt-line principle of operating to allow any railroad carriers to enter on the tracks laid on any pier must actually be in force and be subject to the control of the port authorities, a control which can compel that principle to be enforced.

This is true with every other unit and element connected with the operation of a port. The regulation of privately owned water-front facilities also enters into this question of port control, particularly in some of the older ports. Sometimes attempts are made to utilize tide-water frontage for other than shipping purposes. Unquestionably, the primary interest of any community in its port lies in its shipping development, and, therefore, docks for ships are of the foremost consideration. Water-front structures for private storage purposes or other uses should not be permitted.

Recently, in Philadelphia, Pa., a decision was made denying the plea of a large corporation for a permit to extend a pier to the pierhead line where the dock or water space on each side of the pier was less than 70 ft. and the pier extension only 55 ft. wide. In addition, it was proposed to erect a superstructure covering the entire area of the pier without any provision whatever for an apron. The proposed structure was also of such character as to indicate clearly its intended use as that for storage and manufacturing, or a combina-

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tion of both, rather than for shipping or commercial purposes. The absence of any provision for cleats, bollards, or any other means by which a ship could be made fast, confirmed this belief. Furthermore, the company entered into an agreement with the adjoining property owner by which it agreed that it would not dock any craft of any kind or description along one side of the pier, all of which indicated to the Dock Department that such a structure was in violation of the laws now governing the port. There was involved in this application serious questions—the extension, for instance, of a pier of inadequate width and inadequate water space and the apparent desire on the part of the applicant to secure for manufacturing and storage purposes an area located on the recognized property of the Commonwealth of Pennsylvania.

The Director of the Department of Wharves, Docks, and Ferries, in denying the application, stated that he was alive to the importance of the industry in question to the Port of Philadelphia, and recognized that every facility should be afforded for its further development, and although he recognized the fact that this particular corporation was handicapped for storage room, he was also mindful of the responsibility resting on him as an agent of the State to dispense its bounty and felt that the time was opportune for calling a halt on attempts to secure by license the use of State property for private purposes. In this case, the State property referred to, is the bed of the river between the bulkhead and pierhead lines, on which structures for maritime purposes only can be built.

If sustained by the Courts, this decision will have a far-reaching effect in advancing the development and improvement of the Port of Philadelphia, in that, as pointed out in the opinion, the sanctioning of a non-maritime structure could not be otherwise construed than inimical to the development and best interests of the port.

Attention is called to this decision to show the importance of the control of a port authority over privately owned port facilities and also the fact that the port authority will be less handicapped in its improvement and development programme when it becomes the owner of the riparian rights along the entire shore line.

There are many such problems confronting harbor authorities in their efforts to improve and develop the facilities of a port and very often they are helpless to correct inefficiencies, due to the lack of authority to control all the elements that enter into questions of port government. In one American port, some of the laws regulating port matters were enacted more than a century ago and have not been modified in accordance with the advanced and changed methods of the shipping business.

To reap the full benefits of the improvements and developments that may be made to the harbors, or even properly to carry on successfully the actual construction work of improvement and development, the port authority must have the power that will enable it absolutely to control and regulate each and every element effecting the operation, directly or indirectly, of the port as a unit or a series of units. Its work must be separated absolutely from politics and handled exclusively as a business proposition and it must not be organized and established so that a periodical change of directing heads will

cause a periodical change of policy. Continuity of policy must be preserved. When a comprehensive plan of improvement and development is approved and adopted, the work should be carried on and not fail through lack of authority by the port body. When the port commissioners, or the authorities with whom the control of the port are intrusted, are given powers that will enable them to control or regulate every question concerning the functioning of a port as a whole, the improvement and development of that port will then be much simplified and the port engineer will be much less handicapped in solving his problem of how to produce efficient terminal facilities.

PORT PROBLEMS IN NEW YORK

By John A. Bensel,* Past-President, Am. Soc. C. E.

The subject under discussion has been announced as "National Port Problems". As far as the speaker knows there are no National ports in the United States and, therefore, there are no National port problems, unless problems of immigration or Custom House operation are considered. These are the only National operations in American ports, but as they are not of direct concern to engineers, it does not seem probable that they come under the subject.

On account of these facts, the speaker proposes to discuss some of the problems affecting the Port of New York, as he sees them.

The first question which naturally arises is, what are the problems that affect the Port of New York and thus endeavor to find how they can be solved, if they need solution. At this point, it would appear that one enters into a land of mystery, but the guides are numerous and so are the solutions.

It is quite evident that to many the most important problem in the Port of New York is how to get rid of the water. Many solutions of this problem have been proposed, but probably the one which has occurred to one engineer —of simply filling in one or two of the rivers and a portion of the Bay, all of which appear to be entirely superfluous—is the simplest and easiest to understand, and to him at least the problem is solved.

Other schemes to eliminate the water appear in two quite different forms, one of connecting the mainland of New Jersey with Manhattan Island by means of a gigantic bridge and the other of establishing an indefinite number of small-bore tunnels, built far below the street grade, on Manhattan Island.

The public must be somewhat confused by the solutions proposed, one of which would make a track yard on the most expensive real estate in the world and the other would deposit the freight destined for Manhattan at locations where it is difficult to conceive that it could reach the consignee. In neither of these proposals is an explanation given of the reason for them.

The Port of New York is apparently holding its position eminently well. It does more business than its ancient rivals, London and Liverpool, do in combination, and practically one-half the trade of the country goes through its gates. It is not shown that freight movement would be any cheaper, nor would it by any possibility be any more rapid, and the question naturally arises, "why bother?"

In this connection, nearly thirty-five years ago, certain eminent engineers reported on the problem of how to increase the commerce of the port, and arrived at the novel conclusion that a four-track surface railroad with a track connection to every pier was the solution. Had this joke been perpetrated, the water-front would have been rendered inaccessible and the capacity of the port curtailed.

The great ports of the world owe their importance, not so much to their geographical layout, as to the commercial activity of the back country, and

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the Port of New York is no exception to this rule. Had there not been a natural port and harbor, the commercial necessities of the great State of New York would doubtless have produced one, for human ingenuity has known no limit under the pressure of commercial necessity. New York is one of the few great ports located on the sea and, therefore, escapes many of the problems affecting several of the other principal ports of this country and most of the famous ports of Northern Europe. To note some of the problems that New York has escaped, a contemplation of some of the difficulties that have been overcome, is reassuring; Philadelphia is 94 miles from the sea; New Orleans and Montreal are both examples of river development; Seattle and Portland are also examples of port development under adverse conditions; while, in Europe, the situation of the great ports—Liverpool, Manchester, London, Antwerp, Rotterdam, and Hamburg—brings to mind the natural obstacles that have been overcome in their development. All this suggests that, perhaps, the problems of the Port of New York, or of any other port, are not really National; they are local issues or problems. This was said once about the tariff, and although it took about twenty-five years to demonstrate it, it was finally generally admitted to be true. It is to be hoped that Americans learn more quickly now.

Most problems mentioned in regard to freight movements in the port, pertain to the stated necessity of placing the steamship and the car in direct contact, a matter that is not at all difficult when desirable. Many localities are available on the water-front of more than 500 miles, and the problem only becomes complicated when an endeavor is made to move freight cars around in congested localities into positions that are clearly foreign to their purpose or design. All the New Jersey portion of the water-front of the port is easily adaptable to such a treatment, if desired. The speaker recalls that when the so-called Chelsea improvements were built, none of the great trans-atlantic lines would allow railroad tracks on the piers, although the freight tracks of the New York Central Railroad were laid on the marginal street directly in front of the piers.

It is certain that quite a new type of engineer has appeared, a type that would aid any scheme provided there were sufficient funds ready for dispersal. Reference is not made to natural differences of opinion which must always develop in professional activities, but this new type only endeavors to silence the naturally curious by vociferous assertions of what they know, given, of course, without figures, and it would appear to be clearly propaganda for some hidden interest or the development of some commercial scheme. It is certainly a problem to obtain an unbiased opinion, although the experts for both terminals and ports were never more numerous. Bred to a certain extent by the conditions of war work, they are found in every problem, self-elected and ready to advocate almost any scheme for the expenditure of public funds.

The older methods of proving one's point of view must be restored if the engineer is to take a professional stand before the public. Many of the so-called problems are invented in order that the self-constituted expert or authority may get "his place in the sun", but the time must come when proper

analysis becomes the vogue. Thus, many so-called problems will disappear and much that is now written may be put away until needed.

The port problems of to-day do not differ materially and may be briefly stated as being concerned to a large extent with the commercial connection between the rail or canal carrier and the transatlantic ship. Commercial needs must dictate in the methods to be used, and no servile imitation of foreign methods will comply. The great systems of many of the foreign ports are not to be reproduced however effective a lot of cargo hoists may look against a sunset sky.

As the speaker sees the great problem of the Port of New York, it concerns the effective use of the water as a means of communication between the railroad terminals on the New Jersey shore and the points of delivery nearest those of consumption on a shore line of nearly 500 miles. For freight, there is nothing so economical as movement over a water lane, since no railroad yard admits of such expeditious and economical movement of freight cars as the slip between two piers, and the solution of many problems concerning freight movement may be best expressed by the term, "use the water". Do not think that it is necessary to go under it or high above it. The oldest means of movement, it still possesses all the advantages that it ever had and also is frictionless. It offers an almost ideal opportunity for movements on its surface.

As an illustration, it might be mentioned that there arrived in the Port of New York, recently, a barge carrying as much grain as could be carried by a full train load of about 70 cars, all within a bulk about 300 ft. in length, 35 ft. beam, and 12 ft. in depth, and this quantity of grain, amounting approximately to 2 300 tons, made the run from Detroit, Mich., to New York City in 8 days. Does not the picture speak louder than any words? Avoid the problems—"use the water".

RELATIONSHIP OF RAIL AND WATER CARRIERS

By William J. Wilgus,* M. Am. Soc. C. E.

In times like the present, when an impoverished world is practicing rigid economy as never before, it is to be expected that trade between nations will flow along paths of least resistance and, hence, through those gateways which, other things being equal, exact the least tribute in tolls and time from the rail and water carriers which meet there for purposes of interchange. This admitted, can it be denied that American engineers owe it to their countrymen to exert every force at their command to point out in what respect the principal ports of the United States are lacking and how they may be bettered? It would seem that the first move in this direction should be the bringing together of the transportation interests on land and sea, with a view to perfecting their related facilities in a manner that will best serve their common good.

The usual pier or quay, after all, is a joint terminal, not only for the ship, but likewise for the railroad and for the motor truck. Unfortunately, too often this common ground is under the exclusive jurisdiction of one of these agencies, with dire results to the others. For instance, a steamship line in the sole possession of a pier will seldom, if ever, provide proper space on it for tracks on which the railroads may place their cars for delivering and receiving freight direct. It may dictate to the railroads, without expense to itself, that the interchange shall be effected through the medium of trucking or of floating equipment with its added cost of breaking bulk and of tortuous water-front and marine operations. This situation which obtains to-day in the greatest American port—New York—was brought about originally by natural conditions which antedated the railroad area, and, later, was perpetuated through the provision in the seaboard rate which requires the railroad to interchange freight at the ship side rather than at the end of the rail haul.

The wastefulness of this process can be best illustrated by comparing it with the confusion, delays, and excessive costs that would reign in a joint railroad terminal where different gauges of the main lines and yard tracks would necessitate the breaking of bulk or transfer of car bodies in transit, within sight of their final destination, a condition too absurd for serious consideration.

The remedy for this manifestly unhappy situation lies in the conversion of port authorities and carriers, both rail and water, to the wisdom, or, rather, strict necessity, of abandoning the stingy pier policy and of building generously proportioned wide piers and quays on which there shall be ample space for transit and storage sheds, motor-truck driveways, and track layouts designed for continuous cargo handling uninterrupted by switching operations. All the great ports of the Old World, and even the principal ones of South America and of Canada, have taken this foresighted course, and in the United States the more progressive ports have rather timorously done likewise. There are, however, many ports, notably New York, where the narrow-pier policy still rules. In planning for the future then, let past faults be remedied and

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wide piers substituted for the inefficient and wasteful narrow, trackless ones which have been inherited from the days of the sailing ship and canal-boat.

Another direction in which there is a dire need for improvement is the more extended use of mechanical equipment for reducing the expense of cargo handling and for expediting the release of ships and cars. For example, during the World War, the American Army in France operated in twenty to thirty ports at which in the last month of the war, November, 1918, there were discharged, by a variety of methods, nearly 1 000 000 tons of supplies at an average rate of 449 tons per ship per day, or, approximately, 1.10 tons per lin. ft. of ship berth per day. At one of these ports, American Bassens, there had been completed at that time a partial installation of electric gantry cranes, which despite many handicaps, such as untrained operators, poor lighting facilities, shortage of cars and stevedores, and insufficient "tuning up", made an average record of 717 tons per ship per day, equal to 1.8 tons per lin. ft. of ship berth. This was in marked contrast to the 371 tons per ship, or 0.9 ton per lin. ft. of ship berth, discharged daily at neighboring berths where non-completion of the gantries made necessary the handling of similar cargoes by ship tackle, not only with less speed, but also with greater damage to the supplies and with much more rehandling between ship, car, and shed by reason of the shorter radius of action of the ship tackle.

The effect of the gantry installation on economy of ship time and car time and on man-power is further illustrated by extracts from test observations taken at the same ports, as shown in Table 1.

TABLE 1.

Berth equipment.	Average tons per hook-hour.	Average tons per man-day (10 hours.)	Time of ship in port, in days.
Gantry crane	14.5	6.0	11
Ship tackle	8.4	3.7	16
Superiority of gantry crane over ship tackle.	73%	62%	31%

It will be seen that in this instance of mixed cargoes ranging from Quartermaster supplies to railroad materials, machinery, camions, guns, and fighting tanks, the modern mechanical equipment, even with the handicaps mentioned, was pronouncedly more efficient than the methods in ordinary use.

It is believed that these war experiences, which were borne out by those of the British Army in France, should be applied to peace-time commercial needs. At least, they are worthy of thought in connection with the planning of labor- and time-saving devices at the National gateways.

As bearing on the wisdom of a wide-visioned improvement of American ports, it will no doubt be of interest to cite the outcome of a recent study for a large port extension, which indicated that, although the widening of the piers for the accommodation of suitable track layouts, cranes, transit sheds, and driveways, would increase the construction cost over narrow piers more than 50% and reduce the linear space for berthing ships, the additional cost of the plant as a whole, including not only the piers but also the ships, cars, and

motor trucks, was only 8%, while the reduction in berthing space was more than offset by hastened speed of cargo handling and, therefore, quickened release of ships and cars. In consequence, it was shown that, with an undiminished annual tonnage capacity, the efficiency of the port would be greatly enhanced and the terminal charges, embracing ship and car demurrage, as well as labor, reduced more than one-third, after making due allowance for the increased fixed charges.

These figures, of course, are not claimed to be applicable to all times and places. They may be said, however, to serve the purpose of directing attention to the outstanding wisdom of carefully analyzing each port problem at which rail and water carriers interchange freight, both as regards adequate track facilities on piers and quays and also near-by, properly laid out, supporting yards for the storing and sorting of cars free from interferences with neighboring operations.

Several other features remain for brief comment. The unification of the management, under joint control, of rail and water terminals in the waterfront zone of each port is essential to a full solution of the port problem. With this, should go a co-ordinating belt line under the jurisdiction of the port management; an "open door" for the vast fleet of independent craft unpossessed of exclusive facilities—leased or owned; and working arrangements with the trunk lines, whereby freight consigned to the port shall be suitably grouped at distant clearing yards for movement direct to the particular location for which it is destined, without the need for intermediate reclassification or breaking bulk. There is much room for improvement through a further restriction of free storage which is now a wasteful burden on the rail carrier. Ample warehouses and open storage areas adjoining the piers and quays, with moderate charges, would go far to decrease this evil.

The argument so far has been from the standpoint of international trade. It should not be forgotten that whatever is done to increase the terminal efficiency of American ports may perhaps some day save the National existence when the stress of war may again tax the ports to their utmost. The lessons of the World War should prompt Americans to cure their ills in time of peace. Not to do so, at a modicum of the cost of a few $40 000 000 battleships, seems most culpable to one who witnessed the saving of Paris and, therefore, the cause of the Allies, through the presence of the outer belt line at that city, and who likewise saw how close the port of New York came to a breakdown, due to congestion and inefficiency, during the darkest days of the war. No American would like to see repeated the predicament of Washington in 1776, when the loss of the Battle of Long Island found him with nothing but rowboats with which hurriedly to rescue his army.

Relief in all these particulars would appear to be hopeless unless some competent central governmental agency, clothed with power, shall vigorously take up the problem in all its commercial and military phases, with the determination promptly to find and enforce the application of remedies. It is idle to expect that the ship interests and the many railroads will make any substantial united progress in that direction. Under Section 500 of the Transportation Act of 1920, the duty is placed on the Secretary of War to investigate this

problem in harmony with the declared policy of Congress "to promote, encourage, and develop water transportation service and facilities in connection with the commerce of the United States, and to foster and preserve in full vigor both rail and water transportation." Along with this, it is the duty, in the interest of the country at large, of the Interstate Commerce Commission now called on to underwrite the return on railroad investments, to look into and correct matters that affect the operating expenses of the railroads which are burdened with much of the wasteful practices at the ports, from which waste, the water carriers, often flying a foreign flag, are reaping the benefit. Then, it should not be forgotten that, Americans as a people, are vitally interested in the provision of efficient ocean port terminals for their gigantic merchant marine, as to which it is the duty of the Shipping Board to take action.

Can the Society perform a higher public service than to press on the Secretary of War the need for moving promptly and effectively in accordance with the Transportation Act of 1920, so that henceforth joint water and rail port terminals shall have planned and constructed in a manner that will best serve the country from both the military and commercial standpoints, (1) wide piers for the closest possible liaison between land and water carriers; (2) the best of mechanical devices for the economical and speedy transshipment of freight; (3) unified management of all transportation facilities within the water-front and contiguous zones at each port, including a co-ordinating belt-line railway; and (4) co-operation between the trunk lines, the ship interests, and the port managements for cheapening and expediting the interchange of freight and passengers at the ports? By seizing this opportunity for public service now, engineers may be instrumental in forestalling the future creation of port facilities on plans adverse to the true interest of the Nation.

In order that such a movement may be started, the speaker respectfully suggests to the Board of Direction of the Society that the question of the relationship of water and rail carriers at the ports be taken up with other National engineering societies, with a view to its forceful presentation, primarily to the Secretary of War, on whom the responsibility of investigating this matter is placed by law, and, secondarily, to the Interstate Commerce Commission and to the Shipping Board.

PORT ADMINISTRATION

By B. F. Cresson, Jr.,* M. Am. Soc. C. E.

Great ports cannot reach the condition of maximum efficiency, without some central co-ordinating authority to direct them, any more than any great business enterprise. It is no more reasonable to expect that a port can develop and function properly by leaving its development and operation in the hands of transportation companies, than to expect that the Pennsylvania Railroad could be operated efficiently with a separate board of executives and a separate policy in Philadelphia, in New York, in Chicago, and in Pittsburgh; nor if that great system had separate executives to handle its line haul, yard operations, passenger business, freight business, freight-houses, and passenger stations.

A central directing body is as necessary to the proper development of ports as a body of directors to the development of a railroad system. Without either, the situation is entirely against the modern principles of business; it is uncertain in its results and not in accord with modern civilization.

Time was in the history of practically every great port when the movement of commerce was simple; when there were comparatively few carriers, little competition, and plenty of room to expand; when commerce was attracted by natural advantages; where ships could dock and transfer their cargoes to land carriers with the greatest ease. It made little difference physically with the business of the port in those days whether docks were built, or quays or moles constructed; whether piers extended into the fairway, or whether canals were dug into the land. The important matter was to get contact between deep water and land in the easiest manner.

Before the creation of the great transportation systems, before the advent of the railroads with their powerful interests extending throughout the backlands, before the era of modern finance and business, the need of port administration as it is viewed to-day, did not exist.

In the days of ancient Tyre, land transportation was by wagon, or by skid drawn by man-power, horse, or oxen. The ships that sailed the seas were driven by the wind or by man-power at the oars. The cargo was handled into and out of the ship by men at windlasses; the situation was simple.

These conditions obtained, generally, at Venice, a great port in its prime, and at Amsterdam, which, prior to the supremacy of London, was an all-important port. These ports were laid out for sailing ships and for river and canal barges, and, even in London and in some of the older ports in Europe and in the United States, the early facilities as created were for sailing-ship service, without any thought or design for the service of railroads or of freight-handling machinery.

The situation is now far more complicated. With the advent of steam, it was possible to increase the size of ships, their speed, and their equipment. With the advent of the railroads, there arose the necessity of contact between their tracks and the ships. With the growth of machinery, a different design of port facilities became desirable. With the very many interests—financial,

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transportation, operating, political—it has become a prime necessity to have some powerful authority to co-ordinate the facilities and the operations, and it is now an established fact that there must be public control which should extend not only over the harbor waters, but over the lands and the facilities adjacent thereto.

A modern port is made up of a multitude of different operating and administrative units. There are the transportation lines, the railroads, the steamships, the local marine operators, the river boatmen, the dockmen, the truckmen, the warehousemen, and the great financial interests directing private capital into profitable channels—all fighting for position—and not the least of these interests are those political. It seems impossible to divorce port control entirely from politics, and more than one port has suffered by political domination and by the ability abruptly and without redress to change the entire policy of its development and operation. The surest cure against political interference with the administration and development of the port is to appoint the commissioners in authority with overlapping terms of membership.

The public control by a central port authority does not mean the public ownership of port facilities. It does not mean that private investment must be converted into public holdings, nor does it mean that private initiative should not be encouraged and private money expended to produce modern terminal facilities. It does mean, however, that all these things should be harmonized by a degree of public control that will make effective a quicker and cheaper interchange between land and water carriers, will make the most advantageous use of the water-front and the waterways, and will weld the whole port into a terminal unit that will function with the highest degree of efficiency and economy.

The administration, ownership, and control of a port is not necessarily a part of the local government. There are many examples where the water-front of a city is entirely out of the control of the city itself and is under State or Government control, and, in many respects, this appears to be a right principle, as the port itself and the port facilities can be regarded in no case as strictly local facilities. They are in the service, not only of the city, but of the backlands, and, indeed, of a large part of the State and country tributary to the port.

The Port of Montreal is under the Dominion Government; the Port of Boston is under the Commonwealth of Massachusetts; and the Port of San Francisco is under the State of California.

In the Old World, London, is an interesting example of the trend of modernism in ports. For decades the greatest port in the world, handling the greatest tonnage, its supremacy was threatened by Liverpool.

The Port of London was under a number of private companies which owned the water-front and the terminal facilities and which levied from the commerce passing over the docks tolls sufficient to carry the investment and to yield a profit to the investors.

Liverpool was under a public trust made up largely of representatives of those companies using the port, and the returns from the use of the port were

needed only to meet the current expenses of running it, the interest on the public bonds issued to create it, and the credit necessary to enlarge it.

After an examination by a commission and a recommendation, Parliament, realizing the necessity for maintaining the pre-eminence of London, not only created the Port of London Authority with jurisdiction over the port facilities of London, but also authorized the funds necessary to buy out the private interests and place the ownership of the Port of London under a public trust. That this has been successful need not be discussed.

In spite of the World War, London has been proceeding with its programme of creating modern terminal facilities, and under its central control of ownership and operation will develop and prosper increasingly as time goes on.

In the United States, as a rule, private interests have initiated the development of the ports. They have sought to attract and control commerce by creating good facilities. They have acquired and held strategic water-fronts, but with the development of the railroads, and with the growth of business, there has come an appreciation of the necessity for a very marked degree of public control over port development.

The waterways are the public highways to the sea, and, as such, it is to the interest of the public that easy access to them be provided. This can best be done without the necessity of paying private interests for the right of so doing.

Private enterprise, however, need not be abandoned; there is ample room for private terminal works at all the ports, and they should be encouraged. Private domination of the water-front, however, should not be permitted, but all private interests should be co-ordinated into a general plan, all lined up in their proper position, with their proper functions to perform.

This discussion would be incomplete without referring to the situation at the Port of New York.

The first ship sailed into New York Harbor nearly 400 years ago, the explorer, Verrazano, being the first known European visitor. More than 300 years ago, Hendrik Hudson sailed up the river that bears his name, and, in 1624, Albany was settled and New York City two years later.

A century ago, there were no railroads. Eighty years ago there were a few railroad lines running short distances back from the shore. The Erie Canal, commencing 100 years ago, virtually marked New York to be the country's chief port.

In 1914, more than 8 500 ships entered and as many more left the port. More than 45 000 000 tons of commerce entered and left the port during that year. More than 200 companies operate ships into and out of the port. The 12 railroads which reach it, carried into, out of, or through, the port in 1914 more than 76 000 000 tons of freight. The value of the foreign commerce of the District of New York for the fiscal year of 1917, including gold and silver and foreign exports, amounted to more than $4 600 000 000.

The Port of New York lies in two States, New York to the east, and New Jersey to the west, with the dividing line down the Hudson River, down New York Bay, through Staten Island Sound, and through Raritan Bay. On one side of the port is the great city of New York which, for 50 years, has had its Dock Board or Dock Commission; on the other side, there are

fifty or more separate municipalities, and, until 1921, there has been no central co-ordinating authority to aid in the development of the port as a unit.

It may be argued that this great business which the Port of New York has handled indicates the absence of any need for central control, but New York has a harbor of unrivaled excellence, with 800 odd miles of good water-front, with an absence of any tidal difficulties, and a noteworthy freedom from fog and ice.

New York in its growth and its establishment as the financial center of the New World, however, is overtaxing many of its port facilities, with the result that costs are excessive, that lack of any joint rail facilities has caused the use of a large amount of valuable water-front for rail purposes, and delays and congestion which have existed here, in addition to costs, have tended to divert commerce to other ports. New York is not selfish; it does not desire business which should go through other ports, but it aims to maintain its own position and to accommodate the commerce which would naturally flow through it by reason of its geographical and financial position.

Realizing the necessity for some form of administration that would encourage the better use of existing port facilities and the development of new ones along lines of greater co-operative efficiency, and after a careful and painstaking examination and study extending over several years, the States of New York and New Jersey enacted laws, and, on April 30th, 1921, representatives of the two States met and signed a compact or agreement, which compact or agreement created the Port of New York District, embracing all of New York City on the east, Yonkers, New Rochelle, and other communities; and on the west extending beyond Paterson, Passaic, and New Brunswick, N. J. This control extends about 25 miles to the north of the City Hall in Manhattan, about 16 miles to the east, 23 miles to the south, and 20 miles to the west.

By the same action which created the Port of New York District, the Port of New York Authority was created with broad powers, which powers can be exercised on the acceptance of a comprehensive plan by the Legislatures of the two States. This plan in conference with the representatives of municipalities, civic organizations, and transportation interests, is now being prepared for submission to the next Legislatures of the two States.

This treaty or compact admirably expresses its purpose in the following words:

"Now, therefore, the said States of New Jersey and New York do supplement and amend the existing agreement of 1834 in the following respects: Article I. They agree to and pledge each with the other, faithful co-operation in the future planning and development of the Port of New York, holding in high trust for the benefit of the nation the special blessings and natural advantages thereof."

This compact or agreement between the two States has received Federal sanction by a joint resolution of the Senate and the House of Representatives in Washington, and by the signature of the President of the United States, on August 23d, 1921.

The Port of New York District and the Port of New York Authority are assured facts, sanctioned by the States and by the Federal Government; and thus, New York, the great port of the world in the volume of its tonnage and its business, has come to a realization of the necessity for port administration extending over its port district, and, in addition, will become able to handle more commerce, more rapidly, more cheaply, and more efficiently.

PIER DESIGNS AS DEVELOPED FROM QUAY DESIGNS

By H. McL. Harding,* Esq.

The purpose of this discussion is to determine the minimum width of piers whereby a constant uniform flow of miscellaneous cargoes through the pier may be accomplished without congestion.

It is generally accepted by terminal engineers that for the discharging and loading of freighters there is greater speed with economy at quays than at piers. Where there is ample room, and the physical conditions are favorable, it is desirable first to construct quays and, when more berths are required, to project piers. For an equivalent time-transferring capacity, a quay will require an investment of much less than half that of a pier.

There is more or less uniformity in the plan design of quays for ocean and Great Lake freighters. Each quay unit of a quay terminal is a little greater in water-frontage length than the longest vessel that may berth there. For ocean freighters this may be taken as 700 ft.

For each 700 ft. of quay length, the superstructure facilities consist of railway tracks, dray ways, paved, open dray areas, two sheds, overhead cranes with traveling conveyor hoists, and various surface carriers.

On this quay unit of 700-ft. length, the two sheds, each 200 ft. long, are placed, leaving 100 ft. of space at each end of the unit and 100 ft. of open space between the sheds. These sheds have a temporary holding capacity of 600 000 cu. ft. gross and 400 000 cu. ft. net, and are covered working areas, only for assortment and distribution, freight not removed within 48 hours being transferred to the warehouse.

It is necessary to have a supporting warehouse into which the goods may be placed at the expiration of 48 hours, the expense of moving to be charged against the goods.

Quay units may be taken as 150 ft. in width which is divided as follows: From the edge of the quay wall to the shed is 50 ft., with two railway tracks; then the shed, 50 ft. wide; and 50 ft. to the rear of the shed, with three railway tracks, making in all five railway tracks per unit. The outer 25 ft. in front of the shed is for dray ways and ship-side approaches and the remaining 25 ft. for the two railway tracks. The width of the shed may be increased to 70 ft., but, with steam railway practice, the narrower width is preferable.

The roof surface which is used chiefly for transfer and not for storage is of sufficient strength to sustain a load of 250 or 300 lb. per sq. ft. and is of sufficient height to admit of 20 ft. of tiering. Racks are often used in high tiering. At Manchester, England, freight is tiered as high as 40 ft. On account of the tiering which is done by overhead cranes and traveling electric hoists and tiering machines, it is necessary to reserve less floor space for longitudinal and transverse movements, than where there are chiefly floor movements. The expense and time for long horizontal floor movements are greater than for short mechanical vertical movements. There are a number of fixed overhead crane tracks perpendicular to the length of the shed and

* New York City.

on these and on the movable cross-tracks travel overhead hoists. These traveling hoists may be operated either from cabs or from the floor.

To the rear of the sheds the three railway tracks are spanned by gantry cranes similar to those in the front of the shed, unless warehouses are constructed, in which case standard bridge cranes supporting revolving gantry gib cranes would be used.

In the 300 ft. of open space on this 700-ft. length of quay is placed bulk freight, lumber, steel girders, and freight which is not affected by the weather. It is not desirable that such freight should pass through the transit shed, or that the expense and delay of lightering or of moving the ship to another berth should be incurred.

Similarly, in other mixed cargoes of which the greater part may be out-of-door freight, an open quay has been deemed necessary, even though there may be quite a volume of general merchandise for which there must be a rehandling. By the use of the open spaces described, the necessity will be obviated at a public terminal of open quays, especially where the cargoes are generally composed of different commodities. Greater speed and economy can be obtained by utilizing these open spaces instead of sheds. There are also fire-risk advantages in not having the sheds continuous over the whole area.

The warehouses of which there may be a large number are not included in the quay unit width, although they are indispensable to any successful terminal.

The aforesaid plan description and the enumeration of the mechanical and other facilities of the quay unit are given in order that it may be understood how the pier is derived from the quay.

The capacity of the quay unit should be measured not by the storage capacity of the unit shed and open spaces, but by the rate of flow of the freight through the unit. The reservoirs for the storage of goods are the warehouses, not the transit sheds. The rate of flow is preferably calculated by cubic feet per unit of time, therefore, the rate of flow is controlled by the volume, not by the weight totals.

A 7 000-ton ship has a net registered capacity of 700 000 cu. ft., or 100 cu. ft. per ton. This 100 cu. ft. per ton is taken as the unit of flow measure instead of the 40 cu. ft. per marine ton, which is much below the average of general merchandise. The 100 cu. ft. per ton is chosen to cover average maximum conditions.

To determine what maximum rate of flow may be required, this 7 000-ton ocean or Great Lakes freighter may have five hatchways or continuous openings. Each hatch will have two winches and in some cases four winches and two cranes. There will be, therefore, ten winches and ten cranes for each ship operation.

The function of the ship's winches is to draw the freight from between decks above the upper deck, where the draft is burtoned to the hook of the fall rope of the gantry crane and swung by the crane to the shore. In loading the ship, winches are used in stowing.

For example, assume a low average of only 60 cu. ft. per draft (1 200 lb.) in discharging and 60 drafts per hour per crane, which would be 36 000 cu. ft. per hour, making 360 000 cu. ft. per day of 10 hours, or 720 000 cu. ft. with two shifts. Although this 60 cu. ft. per draft may seem to be a low average, it is desired to make the estimates conservative.

There is a net holding capacity in both sheds of 400 000 cu. ft., or about 1 day's flow from the ship, and, in addition, about 250 000 cu. ft. in open spaces for coarse freight. The 650 000 cu. ft. makes a wide and deep channel for the flow. To prevent congestion, it is necessary, however, that a portion of the day's flow from the ship be kept moving as much as possible continuously through the shed.

A certain quantity of the inbound cargo will be discharged over the offside of the ship on harbor craft, another portion may be placed directly on cars or other land carriers, and some also will go directly into the warehouses.

The volume of each draft might be taken as 80 cu. ft. instead of 60 cu. ft. There will be variations due to the character of the cargo, but from 60 to 80 cu. ft. is a fair average.

It is evident, however, that provision must be made to care for a flow of from 36 000 to 48 000 cu. ft. per hour, or 360 000 to 480 000 cu. ft. per day of 10 hours. This, however, represents the flow capacity for only one ship at this berth; hence, the quay dimensions of 700 ft. by 150 ft. should take care of the flow of from 360 000 to 480 000 cu. ft. in 10 hours, but with little surplus capacity. If sheds have less height and a greater floor area, there should not be a gross capacity of less than 600 000 cu. ft. In order that the freight movements of lifting, tiering, and depositing within the covered areas may be done mechanically, one story is preferable to two, since there will be required only one overhead installation of machinery, while there would be two for a two-story shed, besides twice the superintending force. Where there are two or more stories, much of the transferring and handling is performed by manual labor.

At Manchester, England, the sheds have a clear height of 50 ft., which with the same ground area would give a capacity of 1 000 000 cu. ft. Freight can be taken down and transferred to any portion of the Trafford Estates (Manchester) for 12 cents per ton, which denotes great speed of movement.

Different figures may be deduced, but those mentioned will form a fairly close average for high speeds of transference and handling by machinery. Sufficient cranes and traveling hoists can be provided to keep up the flow from the gantry jib cranes. If, however, back of this quay there should be dredged a waterway and a berth should be provided for another ship of similar capacity, there is no doubt but that so much cargo flowing on this 150 ft. of width from the ships would result in delay, congestion, and the too often unnecessary detention of the ship.

Also, it is evident that on a pier only 150 ft. wide and 700 ft. long, in the accommodation of two ships, one would be far too narrow for rapid and economical conditions. The 150 ft., therefore, may be regarded as the minimum width of the quay for one full-length ship.

If two sections of a quay, with all facilities and equipment, are placed back to back giving a width of 300 ft., results could be obtained from this pier which would be nearly equivalent to those obtained from two quay sections each 700 ft. To express the plan in another way, one quay section may be regarded as bent back of another quay section, each constituting one-half of the 300-ft. pier.

On a two-unit pier of 1 400 ft., it is evident that there should be four quay sections each 700 ft. long, two sections on each side of the pier, but that each working-pier-berth-section should still be 150 ft. wide.

It is evident that because the freight of the outer sections of the pier has to pass between the two inner sections, an additional strip in the center of the pier may be required. This strip may be of sufficient width for three additional railway tracks, based on the supposition that the six inner tracks of the 700-ft. pier are designed to be worked to capacity. Railway operators know the necessity and great advantage in having ample trackage, but 300 ft. is often of sufficient width for a two-unit pier.

It is to be feared that where in the past the piers have been too narrow, there will come an era when they will be too wide, regardless of the value of the water-frontage or of the investment required.

The following conclusions may be drawn from this discussion:

First.—Whatever the determined minimum working width of a one-unit quay section of one berth and with the capacity and facilities for one full-length ship may be, the pier for two ships should be double the width of the quay and have double the facilities.

Second.—It is desirable to have railway tracks and dray ways in the center of the pier to the rear of and between the sheds, as well as between the sheds and the water's edge.

Third.—Where a pier is several units in length, and provision must be made for through freight movements between the land ways and the outside units, such increase in width should be designed to be in the central space of the pier.

Fourth.—A pier may be regarded as two sections of a quay with all facilities, bent back to back.

DISCUSSION

WILLIAM H. ADAMS,* M. AM. SOC. C. E. (by letter).—The Great Lakes-St. Lawrence Tidewater Association is an organization of formal State commissions, created by Legislative enactment in the States for which the Great Lakes form the natural outlet. Its work is the organization of public sentiment and the creation of public demand that this improvement be made, by means of the assembly and dissemination of information as to the work involved and the advantages which will accrue to the nation and especially to the Great Lakes region. Its power is being derived from the rising pressure of public opinion which is being moulded, not by oratorical fireworks, but by a carefully planned campaign in which the ammunition consists of technical facts, presented in logical order to the world of business and trade. A large part of its work is being done through chambers of commerce which, to-day, are the most effective means by which business men can get action in public affairs.

In 1919 the President of the Detroit Board of Commerce was appointed on the Michigan State Commission and early the same year the Detroit Board constituted a new committee of the Board designated "The Inland Waterways Committee". The writer was asked to assume the chairmanship of this Committee and to form a working committee from the Board's large membership of more than 6 000 business and professional men. As formed, this Committee included engineers, transportation men, real estate men, and plain "business" men.

It was early seen that there would be an immediate connection between the National project for the opening of the St. Lawrence River to ocean navigation and the local problems involved in organizing the Port of Detroit to take care of world commerce. Accordingly, studies were begun and publicity planned to prepare the city for this phase of the matter. In the fall and winter of 1919-20 a series of articles was run in the *Detroiter,* the organ of the Board of Commerce, reaching its membership and also the business centers of other American cities. This series also appeared simultaneously in the *Michigan Manufacturer and Financial Record,* a financial paper with State-wide circulation. These articles described the development of port facilities in other harbor cities, including lake and world ports. They were carefully prepared and intended to be technically correct, but were written for the business man. They emphasized the business aspect of the work. They told of the relative importance of the "hinterland" as a possible market, and gave records of commerce running back several years. Where available, figures on the cost of port development were given, and comparisons were freely drawn with the facilities for commerce at Detroit. This series ran for about six months. The articles were illustrated, to some extent, and were widely copied in trade publications and house organs. They inspired frequent editorial comment in Detroit and Michigan newspapers.

* Cons. Engr., Detroit, Mich.

Sandwiched into this series was the National publicity for the St. Lawrence Project. All National meetings, conferences, and conventions were carefully reported. Each such event is in large measure a repetition of former meetings, that is, little that is new is brought out at any one conference, although the general mass of data may be constantly augmented. What is generally missed by the engineer is that, while another meeting may not bring out any new data, it has afforded a new opportunity for some one to express an opinion, or for a community or organization to take sides, consciously or otherwise. The data may not be, but the promulgation of it is, news, every time it is done.

The faithful following out of this principle results in much repetition of the same or similar information, but it is probable that each publication reaches some new readers. It is a well-known fact, moreover, that the average reader pays only partial attention to what he reads. If a certain address were to be given in ten consecutive weeks, a new press story could be written each time. The curious fact, to the engineer, is that the general public by no means realizes that it is the same address or even the same set of arguments. Only a small part of one's audience interests itself in "arguments" anyway, or could tell why it believes as it does. About all the average man on the street gets out of a press story is that "the well-known Mr. So-and-So addressed the Blank-Blank Society last night on 'Port Improvements at Detroit'", and he says to himself, "What a wide interest there is being created in this matter. There must be something in it." Then, he begins to talk about it, and what is more important, he begins to study how it will affect his own future and, in Detroit, he lays plans to acquire some of this water-front while it is cheap, "which will certainly be valuable", he says, "when these hopes are realized." He finds that some of his present interests will be improved by increased commerce and straightway becomes a booster for the proposed enterprise, although he actually knows little about it that would interest an engineer.

In the spring of 1920 the Board of Commerce began an investigation covering in exhaustive detail the actual interest Detroit and Lower Michigan would have in the proposed ocean route. This involved a considerable personnel of field and office workers under a paid secretary and its results were embodied in a book "Detroit and World Trade".* This volume is actually Detroit's brief for the opening of the St. Lawrence River, and was presented at the hearing before the International Joint Commission in October, 1920.

During the summer of 1921 an important Conference on the St. Lawrence project was held under the auspices of the Detroit Board of Commerce. It was attended by representatives of cities and States from all over the country. A three-day programme was presented, almost entirely technical, covering all phases of the subject. The work of this Conference received wide comment, nearly ten thousand columns of press accounts being noted within the ensuing months. Favorable public sentiment was aroused to a marked degree. This Conference was followed by a storm of requests for speakers at public and semi-public functions throughout the State. Boards of trade in other cities called for addresses; the University of Michigan put on a lecture in its summer school; civic clubs, Rotary Clubs, Stationary clubs, women's clubs,

* Copy on file at the Headquarters of the Society.

engineering societies, labor unions, lodges, and churches—every organization which so multiplies—and divides—the activities of a large city, clamored for speakers.

Each such event was heralded in the press; each one furnished "copy" for a press story afterward. Of course, an effort was made to fit the presentation to the audience. The appeal to real estate men was not the same as the lecture to public school audiences. Such a subject has so many phases, however, that little difficulty is had in adapting the subject-matter to the time and the place.

Following this July conference, it seemed evident that public sentiment was sufficiently aroused so that the main project could not be seriously delayed or blocked. It was decided, therefore, to form a new group for more intensive study of local conditions. In preparation for this phase, the writer attended the Chicago Convention of the American Association of Port Authorities, contributing an address on "Detroit's Port Problems", which was widely printed in technical and business journals and, including illustrations, was generally published in the Detroit press. The new Port Development Committee of the Board of Commerce was formally constituted in January, 1921, with the writer as Chairman, and a small working committee. Engineering societies and the many business clubs of the city were invited to form their own port development committees, the chairmen of which would be invited to serve on the Central Committee of the Board of Commerce. In this way, the interesting discussions of the Committee were carried back to the other groups of business men.

The work of the Port Development Committee during 1921 has been largely clearing the ground for action. An "Act" to create a Joint Port District, including Detroit and a number of down-river municipalities, was drafted and made the subject of several conferences attended by representatives of all the districts involved. This was accompanied by much press comment, the "Act" being printed in full in most cases.

It was early found that constitutional provisions would prevent the organization of such an inclusive port district, and work was begun on a constitutional amendment to remove the ban. With much attendant press comment, this was pushed through the Legislature, at Lansing, and the campaign for ratification is now on. Work is also in progress on a comprehensive "Act" to incorporate the Port District of Detroit, looking ahead to the next session of the Legislature in 1922.

The interest which has been created is so great that there is constant pressure for more news stories and interviews from the press. The Port Development Committees of Detroit are probably averaging at least a column a week in the principal dailies of the city and State in support of port improvement and the Lakes-to-the-Sea Project.

At the request of the Detroit Board of Education, and following a lecture before the assembled principals and assistants of the public schools, a small textbook is in preparation for use in the Seventh and Eighth Grades and in the High Schools. Debates are encouraged in High School circles, and data on both sides are supplied to contestants.

In conclusion, it should be mentioned that both Committees have had the aid of Mr. Tom L. Munger, a veteran newspaper man and writer, as Secretary. He can find "news" in what is to an engineer the baldest statement of well-known facts. Many of the simplest facts of engineering are absolutely unknown to the public. They only need to be related to some subject which has for the moment the public interest, and to be discussed in an interesting way to become "news", not once simply, but over and over again, as long as the public is still interested in the subject to which it may be, temporarily, related. A technical article, written for the engineer and published in an established technical magazine, will appear once only, be read and filed by a few engineers, and forgotten. If written for the general reader and published in the daily press, or in business publications, it will be read by thousands and reprinted for weeks and months. It may become the source for numberless stories by "hack" writers to appear in quasi-technical publications for years. An address by the writer, delivered nearly a year ago, is this month (August, 1921), being republished in a British magazine on the other side of the world.

Finally, newspaper space should not be under-valued. One is tempted to value only the front page as worth while, but, from long experience, it is evident that even the smallest inch of space on the least important page of the daily paper, is read by thousands. There has never been mention of the Port Development work in the daily press, even in the most casual manner, which has not been brought to the writer's attention by readers, many times.

Even the foregoing discussion, void as it may seem of interest to the general reader, will be the subject of paragraphs in the press calling attention to the need of Port Development in Detroit, the fourth city in the United States.

ARTHUR M. SHAW,* M. AM. SOC. C. E. (by letter).—In this discussion a far-sighted policy is advocated, of including in port plans, provision for "secondary" works, such as railway terminal and classification yards, belt-line connections, warehouses, etc., and an attempt is made to show that such facilities, operated as a part of the port works, will assist in avoiding congestion and reduce the delay of railway equipment and of ships.

Three cases are cited by the writer, which show lack of space for such facilities as are suggested, one representing the maximum in present American port development, one presenting more nearly average conditions, and a Central American port which is just beginning to be a factor in world commerce. These three specific cases are chosen as representing conditions which may be found in most American ports.

For commerce to achieve the highest possible degree of efficiency and the greatest growth, facilities must be provided for the prompt, economical, and orderly transfer of commodities from the point of origin to the point of destination. In an effort to improve conditions, a study of the causes of delay, excessive cost, and confusion is most illuminating. Such conditions are not especially apparent during periods of normal or light business, but become painfully obvious when an unusual or abnormal demand is made on

* Cons. Engr., New Orleans, La.

transportation systems and port works. The World War period was, perhaps, too abnormal to serve as a safe index, but the failure to achieve either speed, economy, or orderliness in the handling of the domestic and foreign commerce of the United States during the year that followed the signing of the armistice, would indicate that a study of the underlying causes of the most glaring defects might now be profitable.

The most obvious causes of delays to American commerce during past periods of unusually heavy business may be given as follows:

1.—Limitations of inland transportation systems, due to shortage of rolling stock and other causes.
2.—Congestion at terminals.
3.—Lack of wharfage and lighterage facilities.
4.—Shortage in the supply of ships.

As it is understood that this is to be one of a number of discussions, each of which will deal with some phase of port development or ocean commerce, the writer will confine himself to a discussion of the second point mentioned, although it is apparent that this has a direct bearing on each of the other three points. For the purpose of illustration, as already stated, three ports have been selected, representing what may be considered as the extremes and the mean of American port development, namely, the Ports of New York, New Orleans, La., and Puerto Cortez (Spanish Honduras). New York is far in the lead of all other ports of the New World in volume of business handled, and the list of commodities passing through this port includes practically every article used by civilized man. New Orleans is the second port of the United States in volume of business, but more nearly represents conditions as they exist in a dozen other ports of approximately the same rank. Its business is largely in agricultural products and in oil in bulk. Puerto Cortez has served as a port ever since the landing of the Spanish conquistador from whom it derived its name, but, until the last few years, it has not been prominent commercially.

At each of the ports mentioned, there has been serious congestion at times of heavy demands or there is evidence that such a condition will arise before any great increase in business handled may occur. Most great ports are the product of years of evolution, and any re-designing to meet new conditions must be made subject to the limitations imposed by existing improvements, many of which are not directly related to the functions of a port. On this account, especial (and usually expensive) methods are necessary to secure room for expansion and for the construction of auxiliary port works.

The following is a description of conditions which particularly affect expansion at each of the ports mentioned, with suggested means for overcoming some of the existing or prospective causes of congestion.

Port of New York.—This port has a great water-front, a spacious and well-protected harbor, and some of the most improved facilities for the storage and handling of cargoes. Tributary to the port are some of the most highly developed transportation systems of the world and, still, extreme congestion at the terminal yards, warehouses, and wharves occurs at times of heavy

movement of freight. If additional facilities could be provided for handling rail shipments and for the temporary storage of goods awaiting transportation, a great saving in handling would be effected and the lay-over time of railroad equipment and of ships would be reduced. This increase in facilities would include a belt-line railway connecting with as many trunk lines as practicable, railway classification yards, warehouses with both rail and water connections, and possibly sites for certain classes of industries which can be operated to the best advantage in the vicinity of large centers of population and convenient to facilities for inland and ocean transportation.

Along and near the water-front of New York City, there is no tract of land now available for such development as proposed herein, but, disregarding arbitrary State lines and considering as a logical part of the Port of New York, all that territory which might be used to advantage in the development of the port, there is still available (as far as any prior use is concerned) great areas of land almost ideally situated for the improvements suggested. These lands are known locally as the Jersey Meadows. The writer is not posted as to their present ownership, although it is assumed that they are controlled by individuals or private corporations and that, under proper procedure, they could be acquired for the purpose of public improvements. It is true that the utilization of these areas for the purposes mentioned would require certain improvements such as drainage and protection works, but no engineering problems would be involved, which have not been solved in a number of similar instances. These improvements will doubtless include the erection of levees to prevent tidal overflow, the cutting of diversion ditches to care for the drainage of adjacent high lands, the construction of a complete artificial drainage system (including sub-drains and pumps), and the filling, by hydraulic dredging, of certain limited areas. The drainage and protection of the City of New Orleans, and of various agricultural reclamation projects in the tidal swamps of Lower Louisiana, may be cited as successful examples of artificial drainage under similar conditions, although both the height of storm tides and the rainfall of the Gulf Coast section greatly exceed those of the New York district.

It may be of interest to note that, during certain years, more has been spent per acre in some counties of New Jersey in an effort to control the mosquitoes in the swamp lands than is required for the maintenance of drainage and protection works of a well designed Louisiana reclamation project. As an incident to the utilization of the Jersey Meadows for the purposes mentioned, a permanent solution of the mosquito problem would be reached.

Port of New Orleans.—The congestion at this port has not become as acute as at New York but, in many ways, the problems are similar. The Mississippi River serves as a natural harbor and affords deep-water wharfage for many miles along the city's front, but the demand for additional space is already insistent. The growth of the city was first along the river banks, and existing improvements make the cost of extensive port developments to the rear almost prohibitive. Partial relief will be afforded on the completion of the new "Inner Harbor Canal" which connects, by locks, with the river and which will add about 11 miles of water-frontage to the harbor. A strip 1 000 ft.

wide, along each side of this canal, and extending for the greater part of its length, has been acquired for port purposes, but no provision has been made for securing additional lands which will be required in the efficient operation of this element of the port.

The 1 000-ft. strips referred to should be used for wharves, wharf sheds, and warehouses, and for delivery tracks, but, supporting these facilities, should be distributing yards and other improvements of the nature of those suggested for the Port of New York. Adjacent to the canal are many acres of unimproved or slightly improved lands which could be acquired at this time at a cost that may be considered negligible in comparison with their ultimate value to the port. The city is now served by a publicly owned belt railway which could be extended to afford all the service required for this new development.

Unless a far-sighted policy is adopted, which will provide for the early acquisition of vacant lands in the vicinity of such cities as New Orleans and New York, for the purpose of port development, it will be only a few years when other interests, not directly concerned in commerce, will secure control of the most favorably situated tracts and make more difficult the improvements which are so obviously required.

Port of Puerto Cortez.—This is a port now "in the making", although local tradition maintains that it was the first continental port of call of the Spanish conquistador, Hernando Cortez. Until very recently, the port facilities consisted of a commodious and safe harbor, a decrepit timber wharf, and a single-track railway leading to the interior of the country. Recent improvements include a reinforced concrete wharf, steel wharf sheds, mechanical loading equipment, and a convenient system of tracks for making rail delivery to ship side.

The Cuyamel Fruit Company operates from this port and has been instrumental in securing the improvements mentioned and, also, with the co-operation with the Government of Honduras, is fostering a varied development of the interior. At present, it is probable that bananas form at least 90% of the exports from this section of the country, but it is realized by those who are most interested, that a business built up on this one commodity is standing on a slippery foundation, and a substantial increase is now under way in cattle raising, sugar plantations, and general farming. It is reasonable to expect that, within the next few years, the present facilities of this port will be outgrown. Ample opportunity exists for the extension of the wharves along the harbor front, but of the "hinterland", there is none.

The Town of Puerto Cortez is built along a single street which occupies the crest of a ridge of beach formation. This ridge carries the track of the National Railway of Honduras, a footpath on each side of the track, and, in a few places, is wide enough for buildings fronting on this "street", although most of the buildings rest on made ground or on "stilts". Street traffic is confined to pedestrians, wheelbarrows, and such vehicles (railway velocipedes, push-cars, etc.) as can be run on the railway track. There are no saddle or draft animals in the town.

Back of this strip of high ground is an "impenetrable" swamp, the word "impenetrable" being used in its ordinary sense which is generally understood

as not applying to exceptionally hardy explorers or to almost any engineering party. The problem of expansion of the secondary port works and of the town itself can be solved only by the reclamation of adjacent swamp lands, as has been suggested for the Port of New York.

In each of the ports described, the need is shown of additional space for handling present business and for future growth. Exact parallels will not be found in other localities, but it is believed that, in general, the problems are similar and that their early solution will contribute to the upbuilding of American commerce.

Unless their transportation systems and ports can be relieved of unnecessary handicaps, Americans cannot expect to compete successfully in the handling of the world's commerce. Obstructions not only affect adversely the American merchant marine, but also hinder the development of American industries and will constitute a real menace to American armies in time of war.

T. F. Keller,* M. Am. Soc. C. E. (by letter).—The question under discussion, "National Port Problems", is one which, when studied from the standpoint of one interested in the improvement of the water-front of the City of New York, brings forward the thought that that water-front is a National port problem. The volume of its business and its close relationship with the remainder of the United States makes the problem a National one.

The City of New York has laid down a comprehensive plan for the development of the Port of the City of New York. There has now been advanced a plan, and the initial steps are being taken, to construct a tunnel under the Narrows, between the Boroughs of Richmond and Brooklyn. When completed, this tunnel with its rail connections, will effect the entry of all the transcontinental lines into the Stapleton Development with its 2 000 000 sq. ft. of pier space and 26 000 lin. ft. of berthing space for overseas steamships, will also open up for intensive development the magnificent Jamaica Bay, and will actually bring the railroads to the ocean.

On the northern side of the city, Flushing Bay will be thus made ready for development, and the wisdom and necessity of deepening Hell Gate to 40 ft. at mean low water will have been proven. Incidentally, connection will be made so that the vast undeveloped shores of The Bronx, along the East River, will attract attention for steamship and industrial purposes.

This tunnel and its necessary connections will bring the railroads through the heart of the Greater City, will open for development the outlying sections, all on deep water, and will serve for all time to show that the City of New York with its responsibility to the Nation has not been remiss in developing its resources.

At present, the city has about 85 miles of berthing on the sides of piers. It can be said, without fear of successful contradiction, that the proposed tunnel, with its necessary rail connections, will add a similar amount of berthing to its facilities, bringing sea and land together, and when served with

* Chf. Engr., Dept. of Docks, New York City.

all the cargo-handling equipment demonstrated to be necessary and efficient, the City of New York will have played its part in solving the "National Port Problem."

The stimulus thus given by the premier port of the world cannot but help in urging water-front development of other American ports to the end that the rivalry created will place all these ports on a high plane of efficiency and will solve the National port problem.

Harwood Frost,* Esq. (by letter).—The writer will discuss herein only one of the many topics that come under the subject of "National Port Problems", namely, the necessity of the conservation of labor at terminal ports.

The term "conservation of labor" rather than "saving of labor", is used, as the mention of labor-saving is too frequently and too generally associated with the dismissal of men, the reduction of the force; and, as a consequence, the introduction of so-called labor-saving devices in any kind of a plant has been bitterly opposed by "Labor" under the mistaken idea that their use would deprive many of the men of their jobs. Facts, however, have not borne out this idea, but rather the reverse, as has been fully demonstrated by the history of printing as affected by the development of the linotype machine, and of many other industries as affected by similar time and labor-conserving equipment.

In the past when labor was plentiful and cheap, it was bought and sold as a commodity, with little thought of its conservation in the manner of the conservation of National resources, such as water power, forests, fuel, etc., and little consideration was given to labor by the nation generally as an important part of its resources. "Labor" looked on mechanical material-handling appliances as enemies, and their introduction was always accompanied by fears of strikes or other labor troubles. With an increasing scarcity and cost of common labor, however, a feeling is developing that such machinery may be looked on as an ally, a means of reducing the drudgery of the laborer's work, of conserving his strength, of increasing his efficiency and enabling him to earn a greater income, thus, in turn, improving his social position, giving him a great buying power in the commercial world and generally elevating him to a higher plane of citizenship.

The problem of labor has always been closely associated with the equally important problem of the economic handling of materials; the two problems are of most ancient origin and go hand in hand. There is no doubt that the transport, lifting, and placing of the huge blocks of stone used in the construction of the great Egyptian Pyramids presented serious problems to their builders, or that serious problems were also presented to the builders of the wonderful aqueducts, temples, roads, and other monumental works of Rome and Greece. Yet, with only a rudimentary knowledge of mechanics, and by the use of the simplest mechanical devices, such as the lever, wedge, pulley, etc., those problems were met and successfully overcome. Go as far back as one will into the historic and prehistoric past, and evidences of these associated problems of labor and material handling are found. These problems have ever existed; their solution has formed the basis of comparative civilization;

* Chicago, Ill.

they have had their effects on history, on society, on industry; they have been the determining factors in wars, and have made and unmade nations; and they have regulated commerce on sea and on land.

The same problems of labor and of methods of handling materials still exist. They seriously affect the daily lives of the people; they make for success or failure in industries; and they regulate the prices of commodities. These problems have always existed; their solution only is different. The ancients solved their problems by the use of vast numbers of slaves, and time was no object. There was no thought of the conservation of anything. So with Americans, when labor was plentiful and inexpensive, there was waste and little thought of it as a commodity of value, to be conserved. With the tremendous drafts on the manufacturing and labor resources of the United States caused by the World War, together with congested terminals and transportation difficulties, however, the people were brought to a conscious realization of the value of labor as a basic commodity and, also, that in the utilization of labor-conserving machinery for handling materials, Americans had not kept pace with other countries. In order to keep costs within limits that would leave a fair margin of profit, whether in the handling of goods through a marine terminal or through a manufacturing plant, it was found to be absolutely necessary to conserve the available labor supply and to raise the efficiency of the man-power by the most extensive possible use of mechanical appliances.

Through the necessities of their overseas war operations, Americans have suddenly developed into a more or less maritime people; they have greatly increased the number of their terminal ports; they aim to see the American flag flying on American ships in all parts of the world; in short, the war forced them into the business of ocean transportation and they now desire to compete for the world's trade against older and more experienced maritime nations. They, however, continue to ignore at their terminal ports the very methods and practices that brought about the efficient and profitable operation of foreign ports, and this in the face of the fact that these same foreign ports are gaining a certain amount of their maritime strength and advantage at the expense of Americans, that is, by the use of large quantities of time and labor-conserving equipment produced by American brains and industry. America has invented, and manufactures more modern freight-handling machinery than any other country in the world and uses comparatively less.

There are a number of reasons for this condition, which it is not the writer's purpose to discuss; he desires, however, to trace briefly one of the results of the condition. The waste in labor at American terminals in the handling of freight has been said to be more than $400 000 000 annually—it may be more or less, but, in any case, the figure is sufficiently high to warrant consideration and to impress one with the magnitude and importance of the problem of this wasted effort. This $400 000 000 of wasted effort may be looked on as representing at a fair rate of interest, a labor capital of $8 000 000 000 tied up by American terminal inefficiency; but, also, consider that, in the passage of goods of any kind between producer and consumer, through the hands of jobbers, wholesalers, retailers, etc., any item of waste

is multiplied by each middleman in just the proportion he increases his selling price above his cost, and that a waste at the source will cost the ultimate consumer about four and one-half times the original waste. On this basis, the $400 000 000 of unnecessary terminal waste would increase the cost of goods to the ultimate consumer by nearly $2 000 000 000. Further, since the finished product of one concern is frequently the raw material of another, the cost of production to the second concern is increased by the multiplication method with every handling, just as the price of commodities is to the consumer. This progressive increase in the cost of wasted effort most certainly has a serious effect on two of the most important problems of the day—the cost of living and the cost of production.

It is evident, therefore, that in the handling of the hundreds of millions of tons of miscellaneous freight which flow through the port terminals of the United States, wasteful methods add to the burden of the consumer, while quick and efficient methods reduce this burden.

The general public, however, does not seem to understand this. There are many men, business men, handlers of the essentials of life, as well as the consumers, who fail to comprehend why they should be interested in the terminal affairs of New York, New Orleans, or Seattle. They readily accept an increase in price said to be due to increased labor costs or increased freight rates, but they do not seem to realize that anything which affects handling costs in any one of the large terminals, or of goods anywhere in storage or transfer, becomes a matter of direct concern to each and every one of them, because they have not given consideration to the close and vital relations that exist between general business welfare and transportation service, that is, the various factors involved in the distribution of commodities. Nor do they realize the fact that whenever the cost of any single element of transportation, such as the unloading of boats, piling, or otherwise moving, of goods in storage, or the loading of trucks, rises above a fair normal level, it becomes a tax on business generally, and the ultimate consumer must pay for it.

The elimination of wasted effort in the handling of merchandise at terminal ports is a matter of public concern, it is a problem of the whole people; it is a National problem, affecting every one, in every locality, in every station of life. This burden of unnecessary taxation and waste cannot altogether be eliminated, but it can be materially reduced by the substitution, whenever possible, of machines for men, and thus by reducing this "cost of lifting", the "cost of living" also will be materially reduced. Any general reduction in the cost of moving goods through the terminals is certain to have its effect along the entire line, and the general public, like the ultimate consumer, is certain to share in the savings, as neither the middlemen, nor the transportation companies, nor any other single interest, can appropriate it all.

L. F. Bellinger,* M. Am. Soc. C. E. (by letter).—There has been much in the newspapers recently concerning the old DeWitt Clinton train with its diminutive locomotive and passenger cars of about one-twentieth the weight and capacity of those of the present. This train is really interesting to the

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present generation, because of the small size and small capacity of the passenger coaches and freight cars, namely, twelve passengers per passenger car inside, and only a few thousand pounds per freight car. At present, it would seem to be impossible to develop economical transportation with the numerous carriages of the DeWitt Clinton size required for a given volume of freight. There are many indications, however, that the container or the original package is to be reduced to approximately the size of the DeWitt Clinton freight car. Steel containers of approximately this size are used in the Warrior River Barge Transportation Service, and wooden containers are used to some extent on the railroads. These containers are taken from barges, lifted bodily by derricks, and placed on flat cars which make rather economical the interchange between barge and railroad train. Especially in less than carload lots, freight consigned to individuals is easily stored in these cubicles. If their total weight is limited to about 5 tons, the entire cubicle can be lifted from the flat cars and placed directly on the ordinary 5-ton automobile truck, delivered to the consignee, unloaded, and the container returned to the railroad.

The economical handling and storing of these cubicles is a matter of detail. At present, much of the package freight is unloaded by ordinary hand trucks, which will carry from 50 to 500 lb., from the freight car to its proper place in the freight-house, thence by hand trucks to automobile trucks, thence to the consignee's storehouse. As now used, the cubicles, with proper design, can be handled by motor-driven piling and stacking machines, carried from the freight car, and placed wherever desired in the freight-house if the cubicle is placed on skids in such a way that the stacking machine can take hold beneath. With the cubicle and the motor-driven stacking machine, one man can handle 5 tons on one trip as against the ordinary 500 lb. maximum with the hand truck.

The devices outlined make for cheaper transportation from point of origin to ultimate destination to the consumer. Heretofore, the effort has been made to reduce the cost of rail transportation per ton-mile until the ton-mile cost in the United States is reduced to its lowest limits, provided the large and expensive freight cars are fully loaded. By means of the cubicles a cheaper freight car is used, namely, the flat car, and the cost of transportation, especially through the freight-houses, is very much reduced.

The connection between the preceding discussion and the increase of port facilities in the United States is not at once apparent. Just as the intention in the past has been to reduce the cost of freight per ton-mile, so the intention of port engineers in the past has been to have a port which would be capable of easily receiving the largest deep-draft steamers that have been built. In order to accommodate these much advertised, maximum sized ships, each small port has dreamed of increasing its facilities so that a ship of maximum size could lie at its wharves and, in the immediate future, rival New York in its commerce. It is believed that the time has come to take up the question of the use of the shallow ports of the globe, to utilize them for the benefit of the remainder of the world to distribute the traffic which is now so concentrated

in the deep-draft ports, and thus to reduce the cost of package handling which results in excessive terminal charges and adds so largely to the total cost of distribution paid by the ultimate consumer.

It is well known that the cost during the time a ship is in port is a very material fraction of its cost of operation. It is also well known that any reduction of time in port adds materially to the earning capacity of the ship. To a certain extent the smaller the ship the shorter is the time to unload and the less idle time is necessary in port. Reduction in the size of the ship can be carried so far that the demurrage is small, but the earning capacity of the ship while in transit reaches the vanishing point. On the other hand, the very large ship wastes so much time in port and, at present, requires so long a time to secure a full cargo that the ship of maximum size is now a losing proposition, which is also true of the very small ship.

There are numerous landlocked bays and harbors in the world, which will permit, with very little or no development, barges and ships of drafts of 10 ft. or less. These harbors were of value about the time of Columbus, or earlier. At the present time, they are unused because of the lighterage costs which occur in handling freight with hand trucks or on the shoulders of stevedores. The freight is handled from the freight warehouse ashore to the barges and again from the barges to the holds of ships lying off shore. Thus, the cost of transportation between point of origin and point of ultimate destination is made too great. There are a number of ports in the world which have spent millions of dollars to secure channels and slips alongside piers that will permit of ships of a draft of 30 to 35 ft. or more. The millions of dollars spent in securing facilities for ships of maximum draft come from the ultimate consumer indirectly in the form of taxes, but, nevertheless, it adds to the actual cost of transportation between the point of origin and the point of ultimate destination.

Instead of increasing the depths of channels, instead of increasing the depths at piers, and in spending additional millions for port facilities, it is believed the time has come again to consider the ports which will accommodate ships of medium draft, and to rehabilitate old grass-grown piers of ports which, although not dead, are at least in a comatose state. There should be a distribution of traffic from the present few large deep-water terminals to water terminals of medium depth, just as at present there is a change in the development of a city from the one large loop district, or the single business center, to several well distributed so-called "civic centers", each fairly complete in itself. When this distribution is effected the following advantages are found to exist: Low, fixed, capital charges; low terminal charges; low cost of land; low cost of building materials; low house rents; and low cost of distribution of food products to these small cities, all of which result in a low cost of living. Each advantage in itself bears directly on a lower cost of labor for those living at the "civic centers", or at the water terminals of medium depth.

Wherever there is a low cost of labor and a low cost of building materials there is a chance to utilize, by means of the cubicles mentioned, a low cost of handling exports from small ports through the small or medium sized ships to the consumers in various parts of the world. The utilization of the medium

sized ships through the medium sized ports will conduce to low freight costs from the point of origin to the point of ultimate destination, through the low costs of handling freight at such ports.

Any reduction in cost of production or of transportation will aid in stimulating trade between Europe and the United States. Any stimulus to trade accelerates the circulation of commercial products and gives debtors a chance to pay their debts and to increase the wealth of the world available for business extensions. Furthermore, it conduces to that international good feeling so necessary to restore business confidence and business health. Anything which will relieve Europe of its sufferings from business "gout", will loosen up the "rheumatic joints" of business in the United States.

It is believed, therefore, that attention should be given not so much to expensive port development involving millions, but to the increase in the use of the medium sized ports which will require comparatively little capital and yet will enliven many different sections of the body politic now in need of resuscitation rather than rehabilitation. In a country the inhabitants of which have a steady income, although small, the people are far more satisfied than those of a country or city where high priced wages obtain, but where work is intermittent. The idle time is used by Satan for the purpose of stirring up dissatisfaction, destructive criticism, strikes, and ultimate revolution.

JOHN H. MCCALLUM,* ESQ. (by letter).—The writer is deeply interested in all port development, especially as it affects all ports of the United States.

As the Port of San Francisco is the gateway to the Orient, the California State Harbor Commissioners are very desirous of furnishing it with the most up-to-date and economical equipment and devices for handling such commerce, and any information that can be gathered, which would assist them in furnishing such facilities and equipment, is eagerly sought. Those who are actively engaged in commercial pursuits and who use this port are of the opinion that the facilities are quite up to date.

The writer has been a member of the State Harbor Board for the past ten years and is quite free to admit that whatever success has been obtained for the Harbor of San Francisco is due largely to the spirit of co-operation between the members of the Board and all factors interested in the port.

During the period of reconstruction which began in 1911, the engineers of the Harbor Board were assisted very greatly by an Advisory Board of Engineers of large experience in water-front construction. In 1917, the rules and regulations, including the system of port revenue and penalties for the use of piers beyond the free period, were completely changed, this change being worked out with the co-operation of a committee appointed by the Board, consisting of those vitally interested in the commerce of the port. The new system has worked splendidly, and especially as it applied to penalties, for, as this was in a great measure the suggestion of the users of the port, they were bound thereto. In 1917, the Board also appointed an Advisory Committee of twenty men doing the greatest business through the Port of San Francisco to advise

* President, Board of State Harbor Commrs., San Francisco, Cal.

with it from time to time on vital problems of the port. This Committee is composed of importers and exporters, ship-owners and operators, representatives of the Chamber of Commerce, Warehousemen's and Draymen's Associations, the railroads, etc. The services of this Committee have been invaluable to the Harbor Board, and its unselfish devotion to the interests of the port has been wonderful. The writer is a firm believer in the policy of consulting the interests that are using the port facilities as to its development.

One of the great engineering problems confronting San Francisco and the Bay region is that of constructing a bridge from the San Francisco side to the Alameda County shore. Some study has been given thereto and a report has been made, but the problem seems to be still unsolved.

Frank W. Hodgdon,* M. Am. Soc. C. E. (by letter).—Ports, as usually known, are points on the sea coast at which merchandise can be transhipped from land transportation to sea transportation, or *vice versa.*

In order to have a port, there must be merchandise to be transported and merchants to control it. If there is no merchandise, there can be no port, and if better facilities are furnished at another port, the merchandise will go there and the port with the lesser facilities will disappear as such, as has been the fate of Salem, Mass. In former times, vessel units and cargoes were small, and land transportation was adequate to convey any bulky merchandise only for short distances, so that many small ports existed which have now disappeared. As railroads were built and developed, they took up the distribution of bulky merchandise and delivered it far inland and brought to the sea the products of the interior. At the same time, the vessel units increased in size, with the result that the facilities furnished by the ports had to be radically changed and enlarged, and the assembling of cargo units could only be done economically by reducing the number of ports and concentrating the business at a few large ones at which the railroads could concentrate. This concentration will continue until the amount of business at any port exceeds its capacity, when it will become necessary to divert some of the business to other ports.

In general, merchandise will take the most direct route, other things being equal, but many things happen to change this, such as rates and facilities for handling the merchandise, congestion on the direct route, etc. The facilities furnished by a port are both fixed and variable. The fixed facilities are the channels, wharves, docks, warehouses, etc., the railroads and highways. The variable facilities are the vessels, railroad cars and equipment, and trucks. These are the facilities only to handle the merchandise which is furnished by the merchants who are the real creators and life of the port, and without which it could not exist. Usually, the port itself controls only the channels, wharves, docks, and, sometimes, some of the warehouses, freight-handling equipment, and railroad yard tracks on its own property. All the vessels, railroads, merchandise, trucks, etc., are owned and controlled by others.

From this, it will be seen that the port merely offers an opportunity to carry on a business over which it practically has no control, except to offer the best

* Chf. Engr., Mass. Dept. of Public Works, Div. of Waterways, Boston, Mass.

practicable facilities to induce business to pass through it. The merchants control the business and can practically make or break any port.

Business like natural forces will follow the line of least resistance. A port naturally situated so as to offer the best conditions as to rates and facilities will secure the most business. New York is situated at the mouth of the Hudson River and, with this inland waterway supplemented by the Erie Canal, was especially advantageously located. It thus became the foremost port of America, attracting business from all over the country, even though handicapped by not having the opportunity to run its railroad trains on to the piers and thus be able to handle merchandise directly between cars and vessels.

The large volume of business and the low costs of land transportation induced by the low grades of the railroad and the competition of the inland waterways attracted more merchandise, so that vessels were attracted since they could usually find a cargo waiting, even if advance arrangements had not been made. At smaller ports, however, such spot cargoes cannot generally be found, and if a vessel brought a cargo to one of the ports without having engaged a return cargo in advance, it would have to sail in ballast and seek a cargo at some other port. This would increase the cost of operating the vessel, and this increased cost would have to be made good by increased freight rates when a cargo was secured. This would make the small port less desirable as a shipping point compared with one which could furnish spot cargoes at most times and thus, again, increase the desirability of doing business in the larger port. Large ports have more frequent sailings of regular lines and tramps than the smaller ports, and small shipments will find more frequent opportunities there and less likelihood of delays.

As already stated, in order to be successful, a port must be properly located, with efficient land transportation tributary to it and a back country capable of furnishing sufficient merchandise to load all the vessels frequenting the port and absorbing all the merchandise which is brought to it. The port must be well equipped with channels, wharves, docks, warehouses, etc., properly to handle the merchandise, and must also have merchants ready to handle the business. A port, to be successful, must first have the business and the wharves, channels, and other equipment, and then the ships will come.

The various ports of the world are managed in a number of different ways: Private ownership including ownership by railroad and corporations; and public ownership such as State, city, or other political divisions. These different methods take no cognizance of the merchant who really controls the port by furnishing the business done through it, and who, if he had some interest in the management of the port, would have a great inducement to persuade other merchants to use it and, thereby increasing its business, tend to reduce the cost of sending his own business through it. Also, the merchants would be able to decide with first-hand information to just what extent it was advisable or profitable to install better or additional facilities.

The method adopted by the Port of Liverpool, England, one of the most successful ports of the world, is based on the idea that the merchant who does business through the port is the person who should manage it. The Corporation of the Mersey Dock and Harbor Board is without capital or shares,

and its credit is based on the prosperity of the port and the value of the property owned by the Corporation. The electors who annually elect the officers of the Corporation are the persons who, during the previous year, have paid port and dock charges of not less than £5. In order to be eligible to be a director, a person must have paid not less than £25. The Directors receive no compensation, but it is considered such an honor to be one that the office is always sought for.

By law, the port charges must be fixed to pay the interest, sinking fund, and maintenance. All improvements and extensions must be paid from the proceeds of loans authorized by the Government. The Secretary and Engineer of the Board are the paid executives. In this way, the merchants have control of the port, and it is to their interest to furnish it with as much business as they can, in order to keep the costs down, as the business is done at cost.

One difficulty at some of the Atlantic ports of the United States, is the inequality of rates when part of the wharves are owned by the railroads, and part by the city, State, or private owners. The railroad companies use their wharves as one of the facilities of the railroad, and make no charge either to the vessel bringing freight for the railroad, or to the goods which are sent over it, other than the regular freight rates. That is, there are no charges for wharfage or dockage, except on goods which are shipped to or from the wharf by other means than by the railroad which owns the wharf. Frequently, the charges for freight on goods brought to or from vessels at a railroad wharf, are less than on similar goods brought to the port for local consumption, although the terminal costs of the railroad are greater for the vessel, than for the local business.

Public or private wharves, of course, cannot compete with railroad wharves on their basis, and some means should be found by which charges should be made, by which the actual cost of facilities at a port should be collected on all goods passing through it, and, if possible, this should be made equal at all wharves. In order to do this, the whole port should be under one control and management, either public or under public supervision. The exact form is not necessarily the same in all ports, but some form of operating the whole port, as a unit, seems to be essential.

M. G. Barnes,* M. Am. Soc. C. E. (by letter).—Since 1914, measured by cost of rail transportation, interior points of the United States have been placed twice as far from the seaboard as formerly. At best, the wheat belt of the United States is handicapped in the world markets because of its great distance from the seaboard. Increased rail rates have aggravated this condition of affairs, lowered grain prices on the farm, and placed the American farmer at such a disadvantage as to limit his foreign market and, indeed, foreign grain may be delivered at the seaboard cities at a less cost of haul than from the grain belt of the United States. Since 1914, the cost of transporting wheat by rail to the seaboard has increased from 12 to 15 cents per bushel. The cost to domestic consumers in the Eastern States has increased to even a greater extent.

* Chf. Engr., Div. of Waterways, State of Illinois, Chicago, Ill.

Based on the past ten-year average annual production of 794 000 000 bushels, these higher freight rates represent a tax on the producer and consumer of at least $100 000 000 annually. This would indicate that on a 10% basis, $1 000 000 000 might profitably be spent to reduce the cost of haul on wheat to the pre-war level; and, of course, if wheat could be hauled at pre-war prices, other commodities would profit in like manner.

Fig. 1 shows the location and quantity, or volume, of the surplus wheat of the United States, and its destination, the figures having been compiled from the records of the U. S. Department of Commerce for the past ten years. Although the destination of the export wheat is roughly accurate, the ports from which it is assumed to move are not the actual ones, but more nearly represent the economic lines of movement based on present rail-transportation facilities. During the open season of navigation, part of this wheat travels by rail and water to the Gulf, and a greater part by rail and water to the Eastern States and the Eastern seaboard, but probably not sufficient to change the routing suggested.

Various suggestions have been made for the improvement of transportation facilities and especially as it affects the farmer. The advocates of the St. Lawrence Ship Canal, permitting ocean liners to visit Chicago, Ill., Duluth, Minn., and other Lake ports, believe their waterway is the best solution of the transportation troubles and advocate its completion jointly by the American and Canadian Governments. Those who favor the improvement of the Mississippi River System of Waterways are equally sure that the export wheat should find exit *via* the Gulf ports. Others see relief only in the improvement of rail transportation.

A study of the wheat map of the United States (Fig. 1), may aid in solving this problem. The great surplus wheat States lie on the eastern slope of the Rocky Mountains, and the wheat finds outlet through four main depots—St. Paul, Minn., Omaha, Nebr., Kansas City and St. Louis, Mo. Under the present rail freight structure, the cheapest route to the seaboard is to the south through the Gulf ports. Shippers realize this, as shown by the wheat export figures for the fiscal years ending June 30th, 1920 and 1921. The shipments are:

	1920.	1921.
Through Gulf Ports.....	97 000 000 bushels	160 000 000 bushels
Through Atlantic Ports..	79 000 000 "	92 000 000 "

and this in the face of poorer facilities and shortage of boats at the Gulf ports. The Eastern ports also have the advantage of Lake transportation during part of the season. The increase in shipments through the Gulf ports in 1921 results from increased freight rates and the better supply of boats at these ports.

In any study of economic wheat transportation, one must not lose sight of the under-production that exists in the eastern part of the United States. Turn, now, to the most economic method of supplying these needs. In export rates, St. Louis has the advantage over St. Paul of 10.2 cents per bushel through Gulf ports, and Kansas City enjoys an advantage of 4.8 cents through the same ports. For domestic trade, St. Louis has an advantage over St. Paul

in the New York markets of 12.3 cents per bushel (not shown on Fig. 1), but Kansas City and Omaha have an advantage of only 3 cents per bushel. St. Louis cannot supply the needs. Manifestly, the wheat should come from the St. Paul District, which it does very largely. Moreover, St. Paul is favored with a partial water route *via* the Great Lakes, and, in 1920, there was carried through the "Soo" Canal approximately the northern surplus shown in the Northern States of Montana, North and South Dakota, and Minnesota. Part of the "Soo" traffic, of course, was Canadian wheat. Flour shipped through the "Soo" Canals is annually about sufficient to supply the New York State shortage.

Next, in order, the shortage in the Southern States must be supplied. This can best be done by drawing on the surplus from Nebraska, Iowa, Illinois, Missouri, and Kansas, as shown. These States actually do supply much of this deficiency. This leaves, as exportable, surplus wheat grown in Wyoming, Colorado, Kansas, and Oklahoma, and the extreme Northwest, which properly finds exit as shown. So much for existing conditions. How shall we set out to improve them?

First, let us consider supplying the domestic trade. Reference to the map (Fig. 1), will show that the great deficiency exists in the Northeastern States. The domestic rail rate from St. Paul to New York is 36 cents per bushel, while the domestic rate from St. Louis to New York is 23.7 cents per bushel, giving St. Louis an advantage of 12.3 cents per bushel. This advantage can be more than offset by a greater use of the waterways now available. Large quantities of wheat and flour are transported annually over the Great Lakes from Duluth, Minn., to Buffalo, N. Y. The quantity of flour passing through the "Soo" Canal annually is sufficient to supply the deficiency in New York State. Now that the New York State Barge Canals are completed, shippers should avail themselves of this waterway to distribute wheat and flour from Buffalo eastward. The normal charge for transporting wheat from Duluth to Buffalo is 2 cents per bushel. At the present time, wheat is sent from Chicago to Buffalo at from $1\frac{3}{4}$ to 2 cents per bushel. In pre-war days, flour was shipped from Duluth to Buffalo for the equivalent of about 4 cents per bushel of wheat. In 1920, the rate was 6 cents per bushel. Wheat and flour from the North Central States would reach Duluth by rail at approximately 10 cents per bushel. This would permit shippers to deliver wheat and flour in Buffalo at rates of from 12 to possibly 15 cents per bushel. After it has reached Buffalo, it is in the territory of deficiency, and can be delivered by rail and water from that point. By an intelligent use of the existing waterways on the northern border of the United States, it is quite possible that a rate of 16 cents per bushel can be maintained from the St. Paul District to New York City, which would show an advantage over the St. Louis all-rail rate of approximately 8 cents per bushel. Navigation through the "Soo" Canal is carried on over a period of about 8 months per year. In order to secure the advantages of water transportation throughout the year, it would be necessary, therefore, to supply storage for a four months' consumption of wheat and flour.

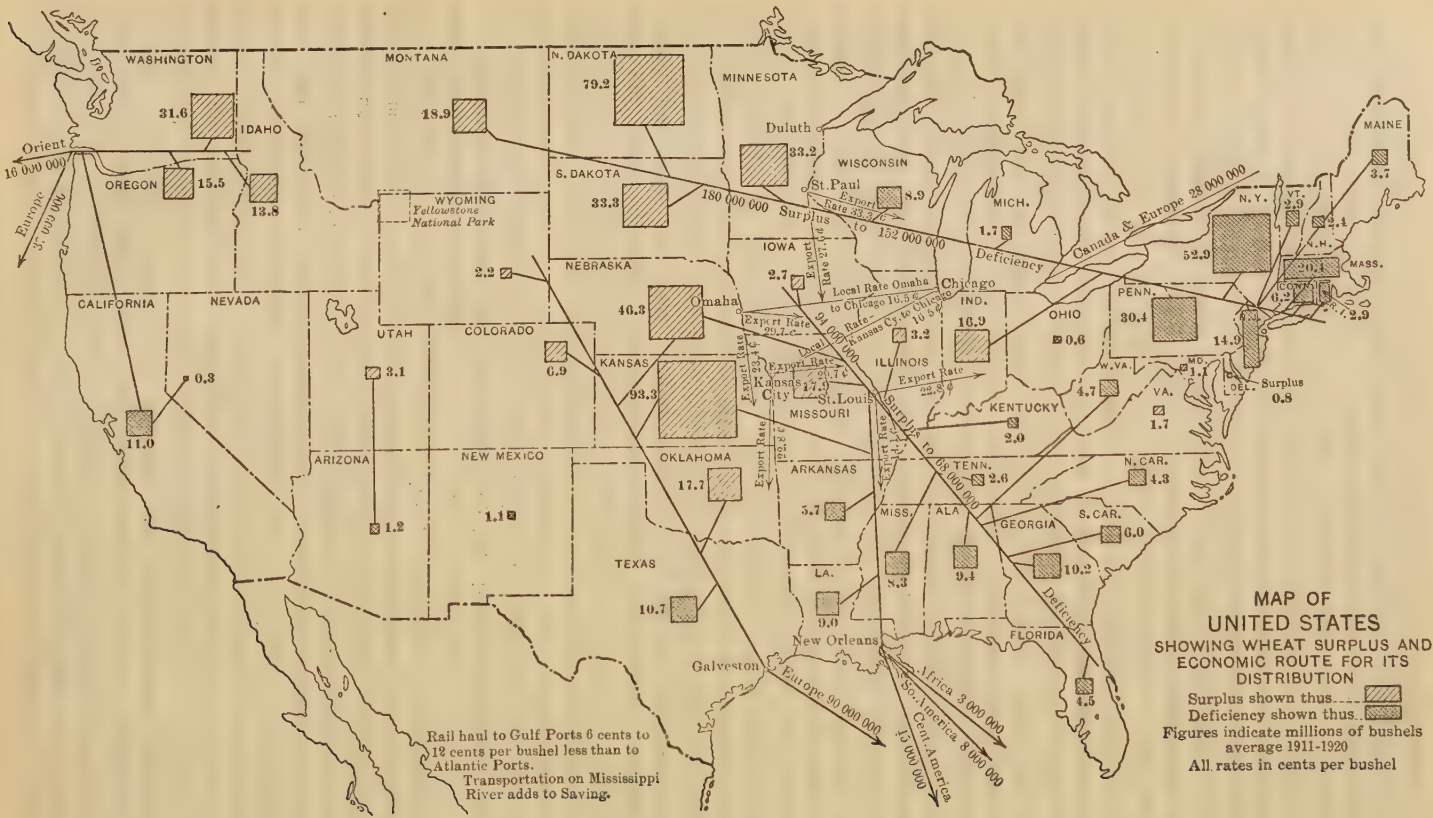
WASHINGTON
31.6
IDAHO
MONTANA
18.9
N. DAKOTA
79.2
MINNESOTA
Duluth
33.2
WISCONSIN
MAINE
3.7
Orient
16 000 000
Europe
35 000 000
OREGON
15.5
13.8
WYOMING
Yellowstone National Park
S. DAKOTA
33.3
180 000 000
Surplus
St. Paul
Export Rate 33.3 c
8.9
to 152 000 000
MICH.
1.7
Deficiency
Canada & Europe 28 000 000
N.Y.
52.9
VT.
2.9
N.H.
2.4
MASS.
0.2
CONN.
0.2
R.I.
2.9
2.2
NEBRASKA
2.7
IOWA
Export Rate 27.1 c
Local Rate Omaha to Chicago 10.5 c
Chicago
IND.
16.9
PENN.
30.4
14.9
CALIFORNIA
NEVADA
UTAH
COLORADO
40.3
Omaha
Export Rate 29.7 c
94 000 000
Kansas City to Chicago 10.6 c
3.2
OHIO
0.6
6.9
KANSAS
93.3
Kansas City
17.9
St. Louis
ILLINOIS
Export Rate 22.8 c
W.VA.
4.7
MD.
1.1
DEL.
0.1
Surplus
0.8
0.3
3.1
11.0
MISSOURI
KENTUCKY
2.0
VA.
1.7
ARIZONA
NEW MEXICO
OKLAHOMA
17.7
ARKANSAS
Export Rate 19.2 c
Surplus to 68 000 000
TENN.
2.6
N. CAR.
4.3
1.2
1.1
5.7
MISS.
ALA.
GEORGIA
S. CAR.
6.0
TEXAS
10.7
LA.
8.3
9.4
10.2
Deficiency
9.0
New Orleans
FLORIDA
Galveston
Europe 90 000 000
Africa 3 000 000
So. America 8 000 000
Cent. America 15 000 000
4.5
Rail haul to Gulf Ports 6 cents to 12 cents per bushel less than to Atlantic Ports.
Transportation on Mississippi River adds to Saving.
MAP OF
UNITED STATES
SHOWING WHEAT SURPLUS AND ECONOMIC ROUTE FOR ITS DISTRIBUTION
Surplus shown thus
Deficiency shown thus
Figures indicate millions of bushels average 1911-1920
All rates in cents per bushel

FIG. 1.

The most feasible means for relief to the Southeastern States is by a more extended use of the rivers of the Mississippi Valley. Wheat and flour can be collected at the important grain and flour centers such as St. Paul, Kansas City, and St. Louis, floated down the Mississippi and Missouri Rivers, and up the Ohio, Tennessee, Cumberland, Warrior, and other streams, to points of consumption. The Federal Government maintains a boat line between New Orleans, La., and St. Louis, which now quotes rates 20% below the rail rates. With the new equipment in use on this line during this season, it is reported that a very handsome profit is being earned at these rates. Great improvement can be realized by the construction of adequate terminals and transshipping facilities at various points along the river. Without doubt, if these streams are properly improved and facilities for handling commodities provided equal to that found on the railroads, water rates can be maintained at as low as one-half the rail rates, and probably much below these figures when quantity shipments are made.

Several schemes have been advanced for the improvement of shipping facilities for the export trade. The destination of export wheat is shown on the wheat map, Fig. 1. It will be noted that the great bulk of the export wheat is consumed in European countries. The North Atlantic cities, including Montreal, have an advantage over New Orleans in distance to London and Continental European ports of approximately 1 500 miles. Montreal has a slight advantage over New York in distance to Liverpool, but owing to the more dangerous route down the St. Lawrence River from Montreal, and to the further fact that Montreal is only a seven months' port, New York has the advantage to all European ports. To compensate for the greater distance to Europe, the rates which shippers are now charging from Gulf ports, are from 1½ to 3 cents per bushel more than from North Atlantic points, or an average of about 2 cents per bushel. The export rail rate from St. Paul, Omaha, Kansas City, and St. Louis, through the Gulf ports, is from 5 to 7 cents per bushel less than through the North Atlantic ports. The net result, therefore, is that the Gulf ports have an advantage of from 3 to 5 cents per bushel in shipments to Europe. This advantage is reflected in the increased use of the Gulf ports. The advocates of the St. Lawrence Ship Canal argue that this advantage can be more than overcome if ocean liners can reach Chicago and Duluth. It has been shown previously that all the north central surplus is required for domestic deficiency in the East Central States. This would leave as the only surplus that from such States as Nebraska and Kansas, to be sent by water from Chicago to European points. The local rail rate from Omaha and Kansas City to Chicago is 16½ cents per bushel. After it reaches Chicago, the wheat is still nearly 1 500 miles from tide-water and more than that distance when measured in time of navigation through open water. In order to reach tide-water, it must pass through the new Welland Canal and through the improved St. Lawrence River, passing possibly sixteen locks in all before it reaches Montreal. With a normal rate of 2 cents per bushel from Chicago to Buffalo, it is probable that a rate of at least 3½ cents per bushel would be required to deliver the wheat from Chicago to Montreal or, say, 20 cents per bushel from Omaha and Kansas City to the seaboard. This

would give an apparent advantage over the Gulf ports of a total of 5 cents per bushel—2 cents in ocean haul and 3 cents in rail haul from Omaha and Kansas City to the Gulf ports. This advantage is apparent only. Wheat is not raised in Omaha and Kansas City, but at a great distance westward from these points, which distance does not add materially to the distance to Gulf ports, whereas, it must be added to any movement east through Chicago. Therefore, even with the construction of the St. Lawrence Ship Canal, it is probable that Nebraska and Kansas wheat would find its way south through Galveston and would not all be collected in Omaha and Kansas City, but at such points as Lincoln, Nebr., Topeka, Kans., Wichita, Kans., and other centers in Central and Western Nebraska and Kansas. It must be remembered that distance from Chicago to Continental Europe by water, *via* the St. Lawrence River, is approximately the same as from the Gulf ports to Continental Europe, and the time of passage through the northern route is greater, with more hazards to navigation. Moreover, the northern route is only a seven months' channel, whereas, the southern route is open throughout the year. There is, therefore, a distinct advantage in shipping export grain *via* the southern route.

In order to lower the cost of haul on export grain from Nebraska, Kansas, and Oklahoma, we must either improve rail facilities or make the Mississippi and Missouri Rivers navigable for fleets of large tonnage. There appears to be little chance of lowering the rail rate, although there is a strenuous effort being made now by agricultural interests to that end. The railroad managers are now arguing that any reduction in rates on wheat must be compensated for by additional rates on other commodities. The Rock Island Railroad estimates that if the rail rate on wheat alone is lowered 20%, the loss to that road will be $5 000 000 annually. It will be seen, therefore, that any reduction in the rail rate is simply a shifting of the burden from one commodity to another. Wheat can be floated down the Missouri and Mississippi Rivers at rates far below the existing rail rates and, with a proper development of these streams, a rate at least as low as one-half the present rail rate can be maintained with profit. This would show a saving in cost of haul from Kansas City of approximately 12 cents per bushel. In other words, a water rate could probably be maintained as low as the pre-war rail rate and, with a return to normal conditions, this rate could be still further reduced. There is an increasing demand for mill products for feed to live stock grown in the Central West. The rates should be regulated so as to encourage the grinding of wheat at Omaha, Kansas City, and St. Louis, retaining the mill product for stock purposes and shipping the finished product—flour—abroad.

Viewing the situation as shown from the wheat map (Fig. 1), there does not seem to be any excuse for any export wheat from the Central West crossing the Mississippi River. The rivers should be improved so as to accommodate craft of large tonnage and shallow draft, and terminals, elevators, and flour mills should be developed along the streams to collect and grind this wheat and float it down to New Orleans.

In addition to the saving in cost of haul, the much congested Port of New York would be relieved. The New York foreign commerce is now

approximately 50% of the total foreign commerce of the United States. The congestion is so great as to add materially to the cost of haul. In the natural growth of the country, this congestion will increase, but no matter how great the facilities for added piers and terminals at the port, the collection of this great tonnage is bound to cause congestion, delays in transportation, and added interest on goods in transit.

The conclusions reached from this argument are:

1.—The producers and consumers of the United States have an added burden of $100 000 000 annually on transportation of wheat due to increased rail rates since 1914.

2.—The main economic route and destination of the surplus wheat of the North Central States is *via* the Great Lakes to Buffalo, thence by rail and water to the North Atlantic States.

3.—The South Atlantic and Gulf States, where deficiency occurs, should be supplied from the Central Mississippi Valley States *via* rail and water.

4.—European, African, South American, and Central American export wheat should come from the Central and South Mississippi Valley States and should be shipped *via* rail and water to Gulf ports.

5.—Additional milling facilities should be provided at Omaha, Kansas City, and St. Louis.

6.—The streams of the Mississippi Valley should be improved to afford an adequate navigable channel for the collection and distribution of wheat and other commodities.

7.—Modern terminals along these streams should be provided both for collection and distribution.

8.—Much may be said in favor of the proposed St. Lawrence Ship Canal, but it does not seem that its completion will materially benefit the surplus wheat raisers of America.

The St. Lawrence Ship Canal has many things to commend it, and probably it should be constructed for its general benefit to commerce and for the power that may be developed along its course; but relief to the farmers must be reached through the Southern route.

Nelson P. Lewis,* M. Am. Soc. C. E.—The speaker has heard with great interest what has been said on this subject, and the freedom and frankness of the discussion has appealed to him. It is well for Americans to know what their Canadian neighbors are doing, and it is well to hear a protest against over-centralization and a plea for the smaller ports.

Some hysterical complaints have been heard that the Port of New York is losing its prestige by reason of the fact that the percentage of the foreign commerce of the United States, handled through the Port of New York, has greatly decreased during the last generation or two. It is quite natural that this should be so in view of the development of other parts of the country with seaports located so as to avoid excessive hauls by rail. Had not new commercial ports been developed and had New York's percentage of the country's total commerce been maintained, the congestion due to over-centralization

* Cons. Engr., New York City.

would have been unwholesome and unfortunate. New York Harbor has such great natural advantages that if the port is intelligently organized and adequate facilities are provided, it will retain all the commerce to which it is entitled, and it need not worry about the development of other ports, whether large or small. There is needed in New York a broader conception of the whole problem and an avoidance of provincialism.

T. Howard Barnes,* M. Am. Soc. C. E.—One of the special topics of this discussion, namely, the relation of warehouses to port development, is timely and should be taken up more at length.

The relation is so intimate and the people are so accustomed to having a warehouse of some capacity connected with the quays and piers that they do not stop to analyze the relation of a part to the whole. This relation has been alluded to by several of the speakers, one of whom noted briefly the economy obtained where ample warehouses were so provided. The remarkable economy per linear foot of pier in certain cities has also been mentioned without a full analysis of the reasons therefor. The question is, to what extent is such economy attributable to warehousing facilities on or at the pier?

The American system, or it might be called the primitive system, of building piers is the putting up of a structure for the quick landing of materials with a minimum of concern, at the time, beyond that of getting such materials unloaded and transferred to the shore. Even now such practice may be noted in less developed regions, and it cannot be wholly condemned in view of the uncertainty of what the future will demand. Many times, however, these developments have grown up "Topsy-like" without sufficient regard for future plans, and, thus, the opportunity for making efficient use of the original structure is forfeited.

Perhaps, the best examples of this practice are the old piers lining the North and East Rivers in New York City. At the time the earlier of these were built, there was no great reason for a storage shed more than one story in height to receive the small cargo. Congestion on Manhattan Island was not the serious question it is to-day. Trucks could come and go without the present-day delays, and near-by warehouses for storage were obtainable. To-day, all this is changed, and the speaker believes that it is an economic crime to permit the erection of piers having warehouses less than three or four stories in height.

An example of the efficiency obtained by providing ample wharf shed was illustrated a few years ago in the extension of a pier in a tropical port, which was under the speaker's charge. A large part of the imports for the interior were brought over this pier. Such imports had always been discharged on to cars, switched into the Custom House on shore, and there unloaded and sorted. The new construction provided for ample floor space on the pier, the roofed area being made level with the platform of the cars. The shed structure was provided with doors at all points by which the cargo could be received as fast as the ship's sling could handle it. The time of discharging was cut substantially in two, likewise the expense of handling on shore,

* Cons. Engr., New York City.

resulting not only in economy of handling cost, but in a very considerable saving in breakage.

All this argues in favor of having a clear space within which to receive the ship's cargo, and it must be remembered that a proper ship's tackle with plenty of winches is an effective way of discharging cargo. It must be borne in mind also that delays in discharging have been due more to a lack of opportunity to keep the cargo moving after it has been lifted out than from the fact that the ship's sling is inadequate, and that overhead cranes or other apparatus might be the remedy.

Once the cargo is landed on shore, the flow must be provided for in accordance with the demands, and it is at this stage that cranes and gantries excel. In the crowded situation in New York City, the possibility of warehousing the lighter portion, at least, in the upper stories, the convenience which this storage means to the importer, and the advantage which it affords the ship owners in accumulating cargo for export purposes, all argue most strongly for provision for that "room at the top" which the multiple-story form of construction secures.

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THE CIRCULAR ARCH UNDER NORMAL LOADS

By William Cain,* M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Mensch, B. F. Jakobsen, Fred A. Noetzli, and William Cain.

Synopsis

In the theory of curved dams, the deflection of a horizontal circular arch under normal loads plays an important rôle. The main object of the following investigation is to derive nearly exact formulas for this deflection, as well as formulas for moment, thrust, and shear, both for arches "fixed" (encastré) at the ends and for those free to turn or "hinged" at the abutments. Since the only approximation introduced consists in neglecting the influence of shear on the deformation, the results may be characterized as nearly exact.

It is left to the engineer's judgment to choose the results best adapted to the conditions at the abutments as actually constructed. For thin dams, the theory pertaining to "hinged ends" may be nearer the truth; for thick dams, the hypothesis of "fixed ends" may be more nearly realized. In case anchoring with steel bars to the side-hills is resorted to, the conditions there become more definite.

The formulas being necessarily long, a number of tables of numerical coefficients have been prepared to aid in a quick computation of deflections, or for use where deflections are given by observation and stresses are to be ascertained.

In the analysis of arched dams, the dam is supposed to be divided into a series of horizontal arches, each 1 ft. in depth, and free to move over each other,† and the solution is based on the principle that the deflection of any arch and of the corresponding vertical cantilever at its crown, must be the same. The part of the normal load borne by each arch in turn is thus ascer-

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† On this point, see the writer's discussion of the paper entitled "Gravity and Arch Action in Curved Dams", by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 71.

tained; and it is this load, acting in a radial direction, that figures in all the computations that follow.

Very simple formulas for the deflection of a horizontal arch subjected to normal loads, have been derived by B. A. Smith, M. Am. Soc. C. E.,* and by F. A. Noetzli, Assoc. M. Am. Soc. C. E.†

Using the notation given in connection with Fig. 1, the crown deflection η, can be reduced to the forms:

$$\text{(Smith)}, \quad \eta = \frac{3}{2} \frac{p' r'^2}{E t},$$

$$\text{(Noetzli)}, \quad \eta = 1.56 \frac{p' r r'}{E t}.$$

These formulas were derived, for arches "hinged at the ends", by considering the deformations due only to the circumferential thrust, the influence of bending moments being neglected. They are, consequently, only approximate.

In Table 1 are given the numerical coefficients corresponding to the (nearly) exact solutions of Equations (15) and (19). By comparison, it is seen that the coefficient, 1.56, of Mr. Noetzli's formula, is sufficiently near for thin dams with "hinged ends" for the usual central angles; but for thick dams and small central angles, the approximate solution gives very erroneous results. The differences are still more pronounced when the arch is really "fixed at the ends."

TABLE 1.—Circular Arch Fixed at Ends, Under Normal Loads.

Values of Coefficient c in $\eta = c \left(\frac{p\, r^2}{E\, t}\right) = c \left(\frac{p'\, r\, r'}{E\, t}\right)$.

$2\phi_1$	$\frac{t}{r} = 0.02$	$\frac{t}{r} = 0.06$	$\frac{t}{r} = 0.10$	$\frac{t}{r} = 0.15$	$\frac{t}{r} = 0.20$	$\frac{t}{r} = 0.25$	$\frac{t}{r} = 0.30$
40°	1.708	0.994	0.542	0.287	0.173	0.115	0.081
60°	1.845	1.606	1.277	0.911	0.651	0.477	0.360
90°	1.879	1.828	1.735	1.577	1.400	1.223	1.060
120°	1.894	1.878	1.848	1.794	1.723	1.640	1.549
180°	1.918	1.916	1.911	1.903	1.891	1.877	1.859

Circular Arch Hinged at Ends, Under Normal Loads.

Values of Coefficient c in $\eta = c \left(\frac{p\, r^2}{E\, t}\right) = c \left(\frac{p'\, r\, r'}{E\, t}\right)$.

$2\phi_1$	$\frac{t}{r} = 0.02$	$\frac{t}{r} = 0.06$	$\frac{t}{r} = 0.10$	$\frac{t}{r} = 0.15$	$\frac{t}{r} = 0.20$	$\frac{t}{r} = 0.25$	$\frac{t}{r} = 0.30$
40°	1.541	1.363	1.108	0.811	0.590	0.437	0.332
60°	1.571	1.518	1.453	1.331	1.190	1.048	0.914
90°	1.578	1.568	1.556	1.528	1.490	1.444	1.392
120°	1.593	1.590	1.587	1.578	1.566	1.551	1.533
180°	1.637	1.636	1.635	1.634	1.632	1.630	1.627

For the latter case, the late R. Shirreffs, M. Am. Soc. C. E.‡ derived an approximate formula for deflection which is as complicated as the (nearly)

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

‡ *Transactions*, Am. Soc. C. E., Vol. LIII (December, 1904), pp. 163–166.

exact formula, Equation (15), and differs from it in its results very materially, so that it should be rejected.

In the ordinary theory, where vertical loads only are considered, the influence of moment and thrust can be separately considered; but, for the uniform normal loads, the investigation that follows will show that this separation is inadmissible, since it gives $M = 0$, throughout. Mr. Shirreffs, in his analysis, attempts this separation and likewise ignores the influence of P on bending, and the compression of the arch due to H', which is really the equivalent of $(p\,r - P_0)$, as given later. These considerations will sufficiently account for the difference in the results mentioned.

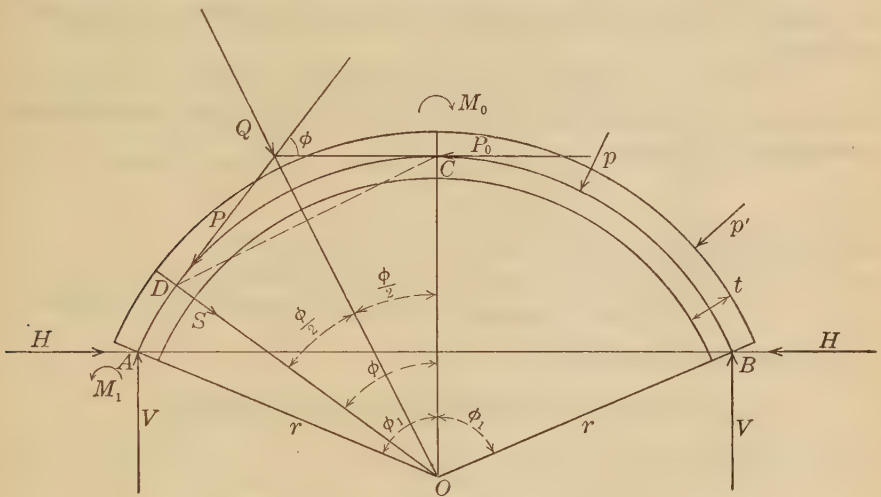

FIG. 1.

CIRCULAR ARCH OF UNIFORM RADIAL THICKNESS, FIXED AT THE ENDS AND SUBJECTED TO A UNIFORM NORMAL RADIAL PRESSURE.

Fig. 1 is supposed to represent a horizontal circular arch, 1 ft. thick perpendicular to the plane of the paper.

Let, t = uniform radial thickness of arch, in feet;
r = radius of center line of arch, in feet;
r' = radius of extrados, in feet;
p' = normal radial pressure, in pounds per square foot, on extrados;
p = normal pressure, in pounds per square foot, on center line $= \dfrac{p'\,r'}{r}$;
ϕ = angle with radius of crown for any point, D;
ϕ_1 = half central angle, $A\,O\,B$;
s = length of arc, $C\,D = r\phi \therefore ds = r\,d\phi$;
E = modulus of elasticity of arch, in pounds per square foot;
M_0 = moment at crown, taken positive clockwise;
P_0 = thrust at crown;
M, P, and S are, respectively, the moment, tangential component of the thrust, and shear at D (r, ϕ).

From symmetry, the thrust, P_0, at the crown, is normal to the radial section there and the shear is zero. At the left abutment, the components of the reaction parallel and perpendicular to $O\,C$ are V and H, and the reaction moment is M_1. These forces V, H, M_1, with P_0, M_0, and the load on the semi-arch, hold the latter in equilibrium, so that it can be treated as a free body.

By Merriman's "Hydraulics",* the component, Q, of the loads on the arch segment, $C\,D$, acting perpendicular to the plane, $C\,D = p\,.\,C\,D = p\,.\,2\,r\,\sin\frac{\phi}{2}$. Similarly,

$$V = p\,r\,\sin\phi_1 \ldots\ldots\ldots\

From
$$\frac{\delta L}{\delta M_0} = 0,$$
$$\frac{r}{E I}\int_0^{\phi_1} M \frac{\delta M}{\delta M_0} d\phi = 0,$$
therefore,
$$\int_0^{\phi_1} M\, d\, \phi = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$
By aid of Equation (3), this reduces to
$$M_0\, \phi_1 + (p\, r - P_0)\, r\, (\phi_1 - \sin\, \phi_1) = 0 \ldots\ldots\ldots\ldots\ldots (7)$$
From
$$\frac{\delta L}{\delta P_0} = 0,$$
$$\frac{1}{I}\int_0^{\phi_1} M \frac{\delta M}{\delta P_0} d\, \phi + \frac{1}{t}\int_0^{\phi_1} P \frac{\delta P}{\delta P_0} d\, \phi = 0.$$
On reducing, utilizing Equation (6), and putting $\frac{I}{t} = k^2$,
$$r\int_0^{\phi_1} M \cos\, \phi\, d\, \phi + k^2 \int_0^{\phi_1} P \cos\, \phi\, d\, \phi = 0 \ldots\ldots\ldots\ldots (8)$$
On substituting the values of M and P, given by Equations (3) and (4), and integrating, recalling that
$$\int_0^{\phi_1} \cos^2 \phi\, d\, \phi = \frac{\phi_1}{2} + \frac{1}{4} \sin\, 2\, \phi_1,$$
$$r\left[M_0 \sin\, \phi_1 + (p\, r - P_0)\, r\, \left(\sin\, \phi_1 - \frac{\phi_1}{2} - \frac{1}{4} \sin\, 2\, \phi_1\right)\right]$$
$$+ k^2 \left[P_0 \left(\frac{\phi_1}{2} + \frac{1}{4}\, \sin\, 2\, \phi_1\right) + p\, r\, \left(\sin\, \phi_1 - \frac{\phi_1}{2} - \frac{1}{4}\, \sin\, 2\, \phi_1\right)\right] = 0 \ldots . (9)$$

To eliminate M_0, multiply Equation (7) by $r \sin\, \phi$, and Equation (9) by ϕ_1, subtract the last equation from the first, multiply by $\frac{2}{r^2}$, and solve for $(pr - P_0)$,
$$(p\, r - P_0) = \frac{p\, r}{D}\, 2\, \phi_1 \sin\, \phi_1 \frac{k^2}{r^2} \ldots\ldots\ldots\ldots\ldots (10)$$
where
$$D = \left(1 + \frac{k^2}{r^2}\right) \phi_1 \left(\phi_1 + \frac{1}{2}\, \sin\, 2\, \phi_1\right) - 2 \sin^2 \phi_1 \ldots\ldots\ldots (11)$$
From Equation (10), P_0 can be at once derived. From Equation (7),
$$M_0 = -(p\, r - P_0)\, r \left(1 - \frac{\sin\, \phi_1}{\phi_1}\right) \ldots\ldots\ldots\ldots\ldots (12)$$
which gives M_0 when $(pr - P_0)$ has been computed from Equation (10). Similarly, from Equations (1) and (2), the components of the reaction at the abutment are ascertained.

If Equation (12) is substituted in Equation (3), we find,
$$M = r\, (p\, r - P_0) \left(\frac{\sin\, \phi_1}{\phi_1} - \cos\, \phi\right) \ldots\ldots\ldots\ldots\ldots (13)$$

giving the moment at any point, D (r, ϕ). The value of P there is given by Equation (4).

The shear, S, on the radial section at D will be taken as positive when directed toward the center. The load on the segment, CD, of the arch, due to the normal pressures, p, contributes to this shear a component parallel to $D\,O = p\,r \sin \phi$.* The component of P_0 parallel to $D\,O = -P_0 \sin \phi$; whence

$$S = (p\,r - P_0) \sin \phi \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

It will be observed from Equations (4), (12), (13), and (14), how persistently the factor $(p\,r - P_0)$ appears; hence, in a numerical example, it is advisable to begin by computing $(p\,r - P_0)$ from Equation (10); whence P_0 can be found, and from Equations (12), (13), (4), and (14), the values of M_0, M, P, and S can then be evaluated. Table 2 (given under "Temperature Stresses"), will prove an aid in the computation of $(p\,r - P_0)$.

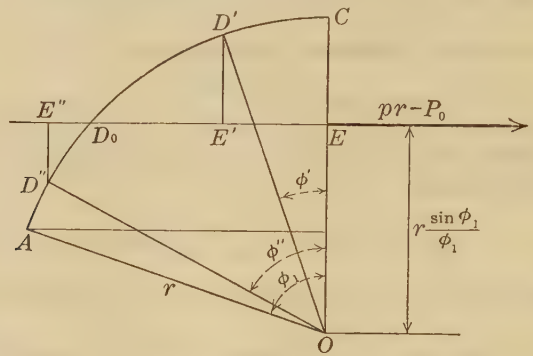

Fig. 2.

The expression for M can be put in a simpler form. In Fig. 2, lay off $(p\,r - P_0)$ as a force acting to the right, perpendicular to the crown radius, $O\,C$, and at a distance $O\,E$ from O equal to $r\,\dfrac{\sin \phi_1}{\phi_1}$, the length (as shown in Mechanics) of the ordinate to the center of gravity of the arc, $A\,C$. Then, from Equation (13), we have,

$$M = (p\,r - P_0)\left(r\,\frac{\sin \phi_1}{\phi_1} - r \cos \phi\right) = (p\,r - P_0)\,(O\,E - r \cos \phi)\ ;$$

therefore, for $\phi = \phi'$, or point D',

$$M = -\,(p\,r - P_0)\,E'\,D'\ ;$$

whereas for D'' $(\phi = \phi'')$,

$$M = +\,(p\,r - P_0)\,D''\,E''.$$

At D_0, where the force line $(pr - P_0)$ meets the arc, $A\,C$, $M = 0$; for points between D_0 and C, M is negative, and decreases numerically in going from C to D_0; for points between D_0 and A, M is positive and increases from zero at D_0 to a maximum at A. From a large scale drawing, the values of $E'\,D'$ and $E''\,D''$ can be scaled off and the values of M quickly computed.

* On the principle (Merriman's "Hydraulics," 8th ed., p. 35) that the component of this load on CD in any direction, equals p times the projection of CD on the perpendicular to this direction, or $p \times r \sin \phi$.

By the theory of the arch, the radial distance from the center of any section, as D', to the center of pressure on that section is equal to $\frac{M}{P}$; hence, since clockwise moments were taken as positive, the line of the centers of pressure meets the crown section on the extrados side of the center line of the arch; it then gradually approaches the center line in going from C to D_0 (Fig. 2); at D_0, it crosses the center line, and from D_0 to A_1, it recedes from it increasingly, lying now on the intrados side, and attains its maximum departure from the center line at A. It is true, from Equation (4), that P increases gradually, in going from the crown to the abutment, but not sufficiently so to invalidate the conclusion, as the following numerical illustrations will show:

1.—Let $t = 4$ ft., $r = 135$ ft., and $\phi_1 = 60°$; then, at the crown, $P_0 = 0.99735\ p\ r$, and $M_0 = -0.061965\ p\ r$; whence, $\frac{M_0}{P_0} = -0.0621$ ft., or the center of pressure at the crown is about 0.06 ft. from the center.

2.—Let $t = 40$ ft., $r = 135$ ft., $\phi_1 = 20°$; then $P_0 = 0.024818\ p\ r$, and $M_0 = -2.657282\ p\ r$; whence, $\frac{M_0}{P_0} = -107.07$ ft., or the center of pressure is 107 ft. from the center of the crown section on the extrados side.

In the first example, the thrust, P_0, is nearly $p\ r$, as given by the cylinder formula; whereas, in the second example, P_0 is only about $2\frac{1}{2}\%$ of $p\ r$. It is evident from this how erroneous it would be to assume the cylinder formula for thick arches. In this second example, the arch action is very small, so that the arch acts nearly as a beam fixed at the ends.

The shear, S, as given by Equation (14), is always positive, or directed toward the center. It is zero at the crown ($\phi = 0$), and attains a maximum at the abutment ($\phi = \phi_1$). Consequently, the thrust, $\sqrt{P^2 + S^2}$, on any radial section is not normal to that section, save at the crown, and its component, S, is directed toward the center. The unit stress at extrados or intrados is given by the usual formulas and diagrams, according as the arch is reinforced or non-reinforced.

For brevity, let the symbol $\doteq$ denote "approaches indefinitely" (as a limit); then from Equations (10) and (12), it is seen that as $t \doteq 0$ (and therefore $k \doteq 0$), $p\ r - P_0 \doteq 0$, or $P_0 \doteq p\ r$ and $M_0 \doteq 0$. Also, from Equations (3) and (4), as $t \doteq 0$, $M \doteq 0$ and $P \doteq p\ r$, the "cylinder formula."

The elastic work due to the axial components, P, is given by the term in P in Equation (5), and its influence in the term involving k^2 in Equation (8) and the following equations; hence, at first glance, it would seem that the effect due to M alone could be found by putting $k = 0$ throughout; but, as shown, this gives $P = p\ r$, $M = 0$ throughout, which is absurd. Thus, the effect of M and P cannot be separated, as is done in the ordinary theory, where vertical loads alone are considered. It appears then, that only a theory like the foregoing, which includes the influence of both M and P throughout, can be expected to effect a solution; and it is on that account, mainly, that Mr. Shirreffs' ingenious solution, already referred to, fails to give reliable results.

Radial Deflection at the Crown, Fixed Ends.

In the case of the arch, Fig. 1, "fixed at the ends", let a small additional load, w, acting in the direction, $C\ O$, be supposed to be applied to the left half arch at the crown. Designating by M' and P', the new moment and thrust at D, we have only to add to M and P, as given by Equations (3) and (4), the proper terms in w. Therefore,

$$M' = M + w\ r \sin \phi \therefore \frac{\delta\ M'}{\delta\ w} = r \sin \phi,$$

$$P' = P + w \sin \phi \therefore \frac{\delta\ P'}{\delta\ w} = \sin \phi$$

As before, the left half arch is treated as a cantilever, and it is supposed to be in equilibrium under the action of the load, w, the couple the moment of which is M'_0, and the thrust P'_0—all acting at the crown—the loads, p, and the consequent reactions at the left abutment.

The elastic internal work of deformation of the cantilever is,

$$L' = \frac{r}{2}\int_0^{\phi_1} \frac{M'^2}{E\ I}\ d\ \phi + \frac{r}{2}\int_0^{\phi_1} \frac{P'^2}{E\ t}\ d\ \phi$$

Therefore, by Castigliano's theorem,* the deflection, η, at the crown, in the direction, CO, is to be found by taking the partial derivative of L' with respect to w and then putting $w = 0$ in the expressions for M' and P', thus reducing them to M and P. Therefore,

$$\eta = \frac{r}{E\ I}\int_0^{\phi_1} M \frac{\delta\ M'}{\delta\ w}\ d\ \phi + \frac{r}{E\ t}\int_0^{\phi_1} P \frac{\delta\ P^1}{\delta\ w}\ d\ \phi$$

$$\frac{E\ I}{r}\ \eta = \int_0^{\phi^1} M\ r \sin \phi\ d\ \phi + k^2 \int_0^{\phi_1} P \sin \phi\ d\ \phi$$

On substituting the values of M and P given by Equations (13) and (4), and integrating, we find,

$$\frac{E\ I}{r}\ \eta = r^2\ (p\ r - P_0)\left[\frac{\sin \phi_1}{\phi_1}\ (1 - \cos \phi_1) - \frac{1}{2}\left(1 + \frac{k^2}{r^2}\right) \sin^2 \phi_1\right]$$
$$+ k^2\ p\ r\ (1 - \cos \phi_1).$$

On substituting the value of $(p\ r - P_0)$ given by Equation (10), and writing,

$$\sin^2 \phi_1 = 1 - \cos^2 \phi_1 = (1 + \cos \phi_1)\ (1 - \cos \phi_1),$$

$$\frac{E\ I}{r}\ \eta = p\ r\ k^2\ (1 - \cos \phi_1)\ \frac{1}{D}\left[2 \sin^2 \phi_1 - \left(1 + \frac{k^2}{r^2}\right)\right.$$

$$\left.\phi_1 \sin \phi_1\ (1 + \cos \phi_1) + D\right].$$

On substituting the value of D given by Equation (11), the last bracket reduces to,

$$\left(1 + \frac{k^2}{r^2}\right)\ \phi_1\ (\phi_1 - \sin \phi_1),$$

* "Systèmes Elastiques", p. 27 or p. 265.

so that, finally, on solving for η, we find, after putting $2 \sin^2 \phi_1 = 1 - \cos 2\phi_1$, $\frac{k^2}{I} = \frac{1}{t}$, and dividing numerator and denominator by $\left(1 + \frac{k^2}{r^2}\right) \phi_1$,

$$\eta = \frac{p\, r^2}{E\, t} \frac{(\phi_1 - \sin \phi_1)(1 - \cos \phi_1)}{\left(\phi_1 + \frac{1}{2} \sin 2\phi_1\right) - \frac{1 - \cos 2\phi_1}{\phi_1 \left(1 + \frac{k^2}{r^2}\right)}} \quad \ldots\ldots\ldots (15)$$

The coefficients, c, of $\frac{p\, r^2}{E\, t}\left(= \frac{p'\, r\, r'}{E\, t}\right)$, have been given in Table 1 for various values of $\frac{t}{r}$ and $2\phi_1$.

In applying this formula, if t and r are in feet and p in pounds per square foot, then E must be expressed in pounds per square foot (not inch). Also, ϕ_1 must be expressed in radians, corresponding to the angle as given in degrees, for which the trigonometric functions are found. Since, generally, $2\phi_1 > 90°$, use, $\sin 2\phi_1 = \sin (180° - 2\phi_1)$, $\cos 2\phi_1 = -\cos (180° - 2\phi_1)$. In numerical computations, such a logarithmic table must be used as to give, at least, three significant figures in the denominator. A 5-place table will usually suffice for $2\phi_1 > 90°$; but for smaller central angles, a 7-place table may be necessary. In fact, for $2\phi_1 = 20°$, an 8-place table was desirable.*

Since $\frac{k^2}{r^2} = \frac{1}{12}\left(\frac{t}{r}\right)^2$, the values of c in Table 1 remain the same, whatever the values of t and r, provided the ratio, $\frac{t}{r}$, remains unaltered. The factor $\left(1 + \frac{k^2}{r^2}\right)$ in Equation (15) can be written 1 for small values of $\frac{t}{r}$ when $2\phi_1 > 120°$, but this approximation would lead to large errors for small central angles. Thus, for $t = 4$, $r = 135$, $2\phi_1 = 20°$, the true value of c is 0.414; whereas, if $\left(1 + \frac{k^2}{r^2}\right)$ is replaced by 1 in Equation (15), the value of c resulting is 1.884. For large values of t $\left(\text{or of } \frac{t}{r}\right)$, the term in k must be retained throughout.

Circular Arch Hinged at Ends.

As before, the arch will be assumed to have a constant thickness, t, and to be subjected to normal loads, p' lb. per sq. ft. on the extrados, or $p = \frac{p'\, r'}{r}$ lb. per sq. ft. along the center line of the arch. The center of the sections at the abutments will be regarded as fixed, but the sections there will be free to turn, or, technically, the arch will be "hinged at the ends." In addition to the previous notation, let (x, y) denote the co-ordinates of D

* An admirably arranged 8-place table is the one by Bauschinger and Peters. It is as convenient to use as a 7-place table.

with respect to A for an origin, x being perpendicular and y parallel to $O\,C$, the crown radius.

The Equations (1) to (5) pertaining to Fig. 1 apply equally to the case of hinged ends, Fig. 3.

Since $M = 0$ at the hinge, $\phi = \phi_1$. Therefore, from Equation (3),

$$M_0 = -(p\,r - P_0)\,r\,(1 - \cos\phi_1)$$

which reduces Equation (3) to,

$$M = -(p\,r - P_0)\,r\,(\cos\phi - \cos\phi_1) = -(p\,r - P_0)\,y \ldots\ldots (16)$$

The horizontal deflection of the crown is zero; whence $\dfrac{\delta\,L}{\delta\,P_0} = 0$, which gives the equation just below Equation (7), or,

$$\int_0^{\phi_1} M\frac{\delta\,M}{\delta\,P_0}\,d\,\phi + k^2\int_0^{\phi_1} P\,\frac{\delta\,P}{\delta\,P_0}\,d\,\phi = 0 \ldots\ldots\ldots (17)$$

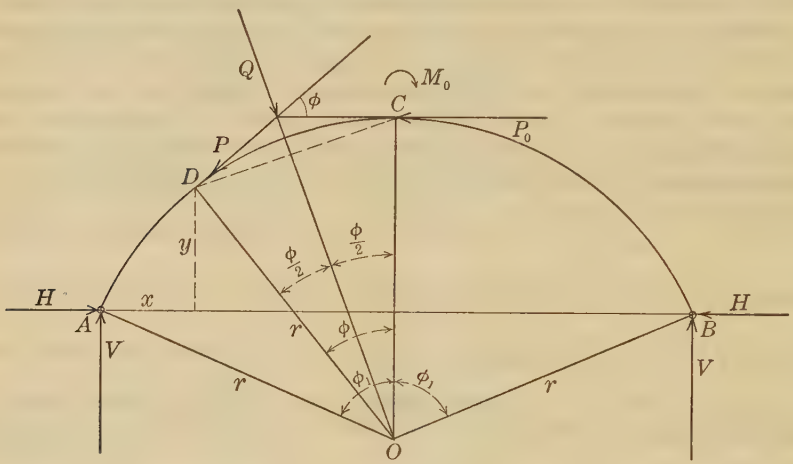

FIG. 3.

Substituting the values of the partial derivatives, integrating, and solving for $(p\,r - P_0)$, we find,

$$p\,r - P_0 = \frac{2\frac{k^2}{r^2}(p\,r)\sin\phi_1}{\phi_1\,(2 + \cos 2\,\phi_1) - \frac{3}{2}\sin 2\,\phi_1 + \frac{k^2}{r^2}\left(\phi_1 + \frac{1}{2}\sin 2\,\phi_1\right)} \ldots (18)$$

from which P_0 can be computed.

After $(p\,r - P_0)$ has been computed from Equation (18), the values of H, M, and P can be at once written down from Equations (2), (3), and (4).

Since $(p\,r - P_0)$ is always positive, by Equation (16), M is always negative, so that,

$$\frac{M}{P} = -\frac{(p\,r - P_0)\,y}{p\,r - (p\,r - P_0)\cos\phi},$$

is negative, or the line of the centers of pressure is everywhere on the extrados side of the center line of the arch ring except at the abutment,

where it passes through the center of the section. This line increases its distance from the arch axis in going from the abutment to the crown. This arch, with normal loading, is exceptional in that the line of the centers of pressure does not cross the arch axis. The result, however, is perfectly consistent with Equation (17), since the first integral in this equation is negative and the second positive, so that the algebraic sum can be zero.

It is very instructive to compare certain results pertaining, respectively, to thin and thick arches. Thus, let $r = 135$ ft., $2\phi_1 = 120$ degrees. Then, for $t = 4$ ft., previous formulas give $P_0 = 0.999534\ p\,r$; $M_0 = -0.03145\ p\,r$.

Therefore,

$$\frac{M_0}{P_0} = -0.031 \text{ ft.};$$

whereas, for $t = 40$ ft., $2\phi_1 = 40°$, we find $P_0 = 0.2\ (p\,r)$; $M_0 = -54.0\ (p\,r)$.

Therefore,

$$\frac{M_0}{P_0} = -270 \text{ ft.}$$

Thus, for $t = 4$ ft., P_0 is nearly equal to $p\,r$, and the center of pressure on the crown section is only 0.031 ft. from its center; whereas, for $t = 40$ ft., P_0 is only 20% of $p\,r$, and the center of pressure on the crown section extended is 270 ft. from the crown, on the extrados side.

It is plain from this that the use of the cylinder formula, $P = p\,r$, for thick arches, will lead to very erroneous results, not only for thrusts and moments, but also for deflections.

Radial Deflection at the Crown, Hinged Ends.

Taking the hinged point, A, Fig. 3, as fixed, regard the hinged end, B, as free to slide on a horizontal plane. Then, if a small radial load, w, is applied at the crown, the reactions due to it, at A and B, will be parallel to $O\,C$, and each will equal $\frac{w}{2}$. The horizontal thrust, H, acting from B toward A, can be supposed to be just sufficient to bring B back to its first position when $w = 0$. The arch is now in equilibrium under the normal loads, p, the reactions, V and H, at the hinged ends, the force, w, at C and the reactions, $\frac{w}{2}$, at A and B. Let M' and P' be the moment and tangential thrust at D corresponding.

The moment, M', at D of all forces and reactions to its right will equal the resisting moment of all forces and reactions on $A\,D$ in magnitude and sign provided the moments of right-handed couples of forces on $A\,D$ are taken as negative, since such moments on $D\,C$ were given the positive sign. Thus, the moment of $\frac{1}{2}\ w$ at A, about D, must be given the minus sign, so that $M' = M - \frac{1}{2}\ w\,x$, where M has the value given by Equation (16).*

* The formula for M was likewise directly proved by considering the forces acting on AD.

Similarly, the tangential component at D acting downward must equal the resisting component acting upward, therefore:

$$M' = M - \frac{1}{2} w x, \text{ and } \frac{\delta M'}{\delta w} = -\frac{1}{2} x = -\frac{1}{2} r (\sin \phi_1 - \sin \phi).$$

$$P' = P + \frac{1}{2} w \sin \phi, \frac{\delta P'}{\delta w} = \frac{1}{2} \sin \phi.$$

If U is the elastic work for the whole arch, then, neglecting the work of shear,

$$U = 2 \left[\frac{1}{2} \int_0^{\phi_1} \frac{M'^2}{E I} r \, d\phi + \frac{1}{2} \int_0^{\phi_1} \frac{P'^2}{E t} r \, d\phi \right];$$

whence the radial displacement, η, at the crown, as $w = 0$, is,

$$\eta = \frac{\delta U}{\delta w} = 2 \int_0^{\phi_1} \frac{M}{E I} \frac{\delta M'}{\delta w} r \, d\phi + 2 \int_0^{\phi_1} \frac{P}{E t} \frac{\delta P'}{\delta w} r \, d\phi.$$

On substituting the values of M and P given by Equations (16) and (4) and the values just given for the partial derivatives, we derive,

$$\frac{E I}{r^3} \eta = \int_0^{\phi_1} (P_0 - p r) (\cos \phi - \cos \phi_1) (\sin \phi - \sin \phi_1) \, d\phi + \frac{k^2}{r^2} \int_0^{\phi_1} \left[(P_0 - p r) \sin \phi \cos \phi + p r \sin \phi \right] d\phi$$

$$= (P_0 - p r) \left[\cos \phi_1 (\cos \phi_1 - 1) - \frac{1}{2} \sin^2 \phi_1 + \phi_1 \sin \phi_1 \cos \phi_1 + \frac{k^2}{r^2} \frac{\sin^2 \phi_1}{2} \right] - \frac{k^2}{r^2} p r (\cos \phi_1 - 1).$$

The value of $(P_0 - p r)$, as given by Equation (18), is now substituted; after which considerable trigonometric and algebraic reduction* finally leads to the following formula for deflection:

$$\eta = p r^2 \frac{1 - \cos \phi_1}{E t B} \left[\sin \phi_1 + \phi_1 (1 - 2 \cos \phi_1) + \frac{k^2}{r^2} (\phi_1 - \sin \phi_1) \right]. \quad (19)$$

where

$$B = \phi_1 (2 + \cos 2\phi_1) - \frac{3}{2} \sin 2\phi_1 + \frac{k^2}{r^2} \left(\phi_1 + \frac{1}{2} \sin 2\phi_1 \right).$$

Writing this in the form, $\eta = c \frac{p r^2}{E t} = c \frac{p' r r'}{E t}$, the coefficients, c, can be computed for varying values of $2\phi_1$ (the central angle) and $\frac{t}{r}$. The results are given in Table 1 and have previously been discussed in connection with those pertaining to certain approximate solutions. The general remarks under Equation (15) apply equally here as to ensuring accuracy in the computations.

* The following formulas were used: $\sin 2\phi_1 = 2 \sin \phi_1 \cos \phi_1$, $2 \cos^2 \phi_1 = 1 + \cos 2\phi_1$. $\sin^2 \phi_1 = (1 + \cos \phi_1)(1 - \cos \phi_1)$. The factor $(1 - \cos \phi_1)$ will be found to be common to the terms in the numerator and can be taken out.

In the theory of curved dams, as given by Mr. B. A. Smith,* the formula for arch deflection involves a constant coefficient, c, so that an average value for the horizontal arches of varying thickness, for the entire height of the dam, will have to be used. This can be taken with sufficient accuracy, for thin dams, from Table 1. For very high dams, in which the thickness near the base is considerable, the coefficients may vary too much to effect a practical solution by use of an average coefficient. For thin dams, the solution is very satisfactory, and it can be effected, not only where the base of the dam is fixed, but likewise where it is simply supported on the foundation. Mr. Noetzli's tentative method† applies only to the case where the dam is fixed at the base. For this case, an average value of c can be used for thin dams; but for very high dams, the values of c corresponding to the varying values of the thickness, t, should be used. To save labor, a graph of the values of c for different heights, should be made. The solution can then be effected by proceeding along the lines indicated by the writer in his discussion‡ of Mr. Noetzli's paper, either for water loads or for temperature changes; the final test being that the deflections of the supposed horizontal arches and the vertical cantilever, at the crown of the arches, shall be the same at the same depth. The solution gives the values of p for any depth, from which moments, thrusts, shears, and stresses at the intrados and extrados can be computed.

Temperature Stresses.

Arch Fixed at the Ends.—If e = expansion per foot for a rise of temperature of 1° Fahr.; then, for t_0 degrees rise for the circular arch above an assumed mean, the thrust, H, acting perpendicular to the crown radial section, is given by the formula,§

$$H = \frac{2 \sin \phi_1}{D_0} \cdot \frac{E\, I\, e\, t_0}{r^2}$$

where

$$D_0 = \left(\phi_1 + \frac{1}{2} \sin 2\, \phi_1\right) \left(1 + \frac{k^2}{r^2}\right) - \frac{1 - \cos 2\, \phi_1}{\phi_1}.$$

Note that D of Equation (11) equals $\phi_1\, D_0$.

The numerical values of $\frac{2 \sin \phi_1}{D_0}$, for various values of $2\, \phi_1$ and $\frac{t}{r}$, are given in Table 2.

The formula for the moment, M, at any point $(r\ \phi)$ of the axis, is

$$M = H\, r \left(\cos \phi - \frac{\sin \phi_1}{\phi_1}\right).$$

It follows, if we conceive H to act to the left at a distance, $r \frac{\sin \phi_1}{\phi_1}$, from the center, Fig. 4, that the moment, M, at a point $(r\ \phi)$, is given by H times

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 2027.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

‡ *Loc. cit.*, p. 71.

§ Professor Church, in "Mechanics of Internal Work", p. 118, derives the same formula, using H and the moment at an abutment as the unknowns. The writer used H and the moment at the crown as the unknowns, giving a shorter solution.

the perpendicular distance from $(r\ \phi)$ to this line of direction of H, the moments being positive (clockwise), for points between this line and the crown, and negative for the remaining points. This line of direction of H passes through the center of gravity of the arc, $A\ C$.

TABLE 2.

$2\phi_1$	$\frac{t}{r} = 0.02$	$\frac{t}{r} = 0.06$	$\frac{t}{r} = 0.10$	$\frac{t}{r} = 0.15$	$\frac{t}{r} = 0.20$	$\frac{t}{r} = 0.25$	$\frac{t}{r} = 0.30$
40°	2 750.4	1 600.0	872.0	461.1	277.9	184.0	130.2
60°	583.7	507.9	403.7	288.4	205.3	150.1	113.2
90°	115.9	112.7	107.0	97.1	86.1	75.0	64.9
120°	86.2	35.9	35.3	34.2	32.8	31.2	29.4
180°	6.7	6.7	6.7	6.6	6.6	6.5	6.5

A remarkable analogy may now be pointed out in the case of water load or normal forces only. Thus, from Equation (10),

$$(p\ r - P_0) = \frac{2 \sin \phi_1}{D_0} \left(p\ r \frac{k^2}{r^2} \right),$$

and it is seen that the values of $\frac{2 \sin \phi_1}{D_0}$ are precisely those given in Table 2; so that the values of $(p\ r - P_0)$ and, thus of P_0, can be at once written down for the values of $2\ \phi_1$ and $\frac{t}{r}$ given.

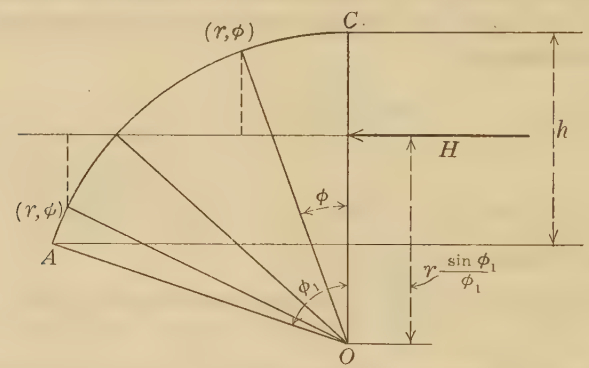

FIG. 4.

Further, if $(p\ r - P_0)$ is taken as a force, with the line of action of H in Fig. 4 only acting to the right, then, since by Equation (13),

$$M = -(p\ r - P_0)\ r \left(\cos \phi - \frac{\sin \phi_1}{\phi_1} \right),$$

the moment at any point $(r\ \phi)$, is equal to $(p\ r - P_0)$ multiplied by the perpendicular distance from $(r\ \phi)$ to the supposed line of action of $(p\ r - P_0)$. This striking property of the circular arch subjected to normal loads, has already been brought out in connection with Fig. 2.

If we let $h = r\ (1 - \cos \phi_1)$ equal the rise of the center line of the arch, then the formula for the crown deflection due to a temperature rise of t_0 degrees Fahr. is,

$$\eta_1 = \frac{h\ e\ t_0 \sin \phi_1}{D_0} \left[(1 + \cos \phi_1) \left(1 + \frac{k^2}{r^2} \right) - 2 \frac{\sin \phi_1}{\phi_1} \right]$$

Arch Hinged at Ends.—For a rise of temperature of t_0 degrees Fahr., the thrust, H, acting inward at an abutment along the span line is,*

$$H = \frac{E\ I}{r^2\ B}\ 2\ e\ t_0 \sin \phi_1$$

where,

$$B = \phi_1\ (2 + \cos 2\ \phi_1\ ) - \frac{3}{2} \sin 2\ \phi_1 + \frac{k^2}{r^2} \left(\phi_1 + \frac{1}{2} \sin 2\ \phi_1 \right)$$

The moment at any point (r, ϕ) of the axis of the arch is,

$$M = H\ r\ (\cos \phi - \cos \phi_1).$$

The radial deflection at the crown is,

$$\eta = \frac{h\ e\ t_0}{B} \left[(1 + \cos \phi_1) \left(2\ \phi_1 \cos \phi_1 + \frac{k^2}{r^2} \sin \phi_1 \right) - \sin \phi_1\ (1 + 3 \cos \phi_1) \right]$$

The derivation of the formulas for deflection is so similar to that for normal forces that it is omitted.

When the Crown Deflection Is Given by Observation.—From formulas given, by expressing H (or the values of $(p\ r - P_0)$ in the case of normal forces) in terms of η $\left(\text{by first writing the ratios, } \frac{H}{\eta}, \text{ or, } \frac{p\ r - P_0}{\eta} \right)$ and substituting in the formula for M, formulas are derived of the form,

$$M = -\ a \left(\frac{E\ t^3}{r\ h} \right) \eta$$

In this formula, η is always positive for water pressure. For a fall of temperature, η is positive. For a rise in temperature, η is negative.

At the crown, $\phi = 0$, $M = M_0$, the values of the coefficients a, when the term in k is ignored, referring to the four cases indicated, are given in Table 3. The results are applicable to the crests of many curved dams,

TABLE 3.

$2\phi_1$	Fixed Ends.		Hinged Ends.	
	Water pressure.	Temperature.	Water pressure.	Temperature.
60°	0.159	0.170	0.096	0.105
90°	0.150	0.180	0.090	0.111
120°	0.138	0.188	0.083	0.121

where t is small and k can be ignored in the formulas. When η is given by observation, M_0 can be computed, if the influence of water pressure and temperature change can be separately estimated.

As dams are usually constructed, the ends are neither fixed nor hinged, so that the engineer is forced to use his judgment in selecting a coefficient,

* Professor Church gives one derivation of this formula in his "Mechanics of Internal Work", p. 109.

which should incline toward that for fixed or hinged ends according to the thickness of the arch and the conditions at the abutments.

Generally, the deflection, η, has been measured for a combined water pressure and change of temperature. If the deflections for each cannot be separately estimated, still some idea of the value of M_0 can be had, for a given $2\ \phi_1$, by taking for a a rough average of the four coefficients, having regard to the conditions at the ends.

To the stresses at the crown corresponding to the value of M_0 must be added the stresses due to P_0 or H. To estimate P_0 (the thrust at the crown due to water pressure), it is necessary to know p, the unit normal pressure carried by the horizontal arch at the crest. No rule can be given for any dam; but, for the Wooling Dam, p is roughly equal to the full water pressure at one-sixth or one-third of the depth of the dam below the crest, according as the dam is fixed or not fixed at the base.* If a complete solution has been made, as in the case of the Wooling Dam, then p *is* known and P_0 can be computed.

* See Tables 5 and 11 of the writer's discussion of Mr. Noetzli's paper on "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 74 and 88. Also, some information can be drawn from Tables 8 and 12, as to estimating H, the thrust due to change of temperature.

DISCUSSION

L. J. Mensch,* M. Am. Soc. C. E. (by letter).—This paper is a valuable addition to the literature on arches, and will dispel speedily and effectively many wrong ideas of designers of past and prospective arch dams.

In his discussion† of the paper by H. Hawgood, M. Am. Soc. C. E., entitled, "Huacal Dam, Sonora, Mexico", the writer has treated the thick arch in a similar manner. He designated the expression, $pr - P_0$, which is really a negative thrust or pull due to the shortening of the arch or a fall of temperature, by T, and believes he has made the action of the shortening of the arch appear more clearly than by compounding the whole action by the method of Castigliano. Since that time, the writer has found that it is not permissible to neglect the effect of shear for the lower arches of a dam and has been agreeably surprised that the formulas for thrust and deflection have not become more complicated and, in some cases, are even simpler than those where shear is neglected. For example, the author's Equation (11) becomes, when the effect of shear is included:

$$D = \left(\frac{1}{4}\sin 2\,\phi_1 + \frac{\phi_1}{2}\right) 2\,\phi_1 + \frac{k^2}{r^2}\left(2\,\phi_1 - \frac{1}{2}\sin 2\,\phi_1\right) 2\,\phi_1 - 2\sin^2\phi_1$$

Any process used to obtain formulas for thrust, moment, and deflections, is always complicated on account of the great length of the equations, the many fractions, plus and minus signs, indices, decimals, and the most skillful mathematician is led into many errors. The writer never considers a new problem solved, until the same result has been obtained by different processes at an interval of a year or more, or when the result is obtained by three persons working independently.

The work of preparing a paper like that of the author is very great, and only those who have tried similar original investigations have any idea of the energy expended on it. It is not surprising, therefore, that such a paper should contain errors. The writer questions Equation (18) and some of the deflection formulas, and many of the figures in Table 1. There seems to be an error in calculating P_0 for the arches 40 ft. thick, in the examples chosen by the author, because the values of $pr - P_0$ ought to be approximately one hundred times greater than for the arches, 4 ft. thick.‡ Arch action, as commonly understood, is small in flat thick arches, and a glance at a flat arch, as shown in Fig. 5, taken from an existing arch dam, will show that the effect of the horizontal components of the water pressure must be very small, and the simplest assumption is that the pressure is taken up by beam action. Can the beam carry this load? This question has not been considered by most writers on the subject. In this case, the water pressure amounts to 10 000 lb. per sq. ft. The corresponding bending moment equals $10\,000 \times \frac{100^2}{24} = 4.17 \times 10^6$ ft-lb. The moment of resistance is $\frac{40^2}{6}$ and the extreme fiber stresses, in pounds per square foot, are $\mp$ 15 000 at the crown and $\mp$ 30 000 at the abutments. It can-

* Gen. Contr., Chicago, Ill.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 610.

‡ This error in the paper has since been corrected.

not be assumed that compression from the arch will reduce these tensile stresses very much, because the arch cannot deflect sufficiently to compress the fibers before they are cracked, or before cracks appear at the construction joints. The beam will show cracks, somewhat similar to those illustrated in Fig. 5. A

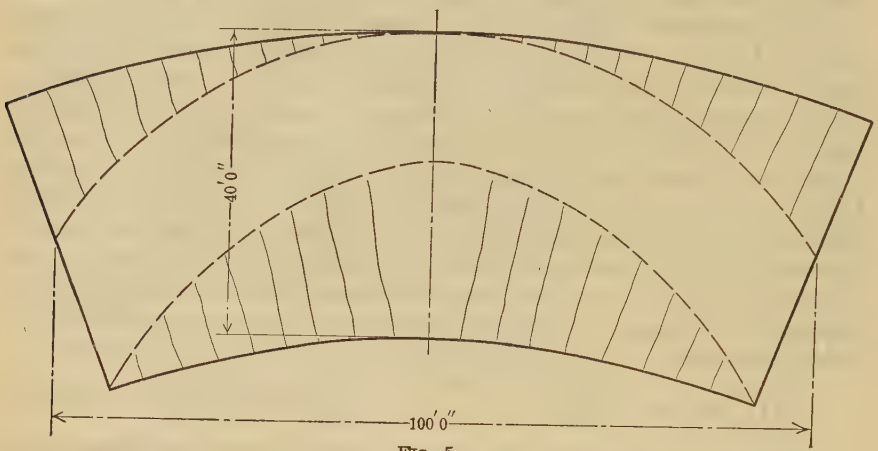

FIG. 5.

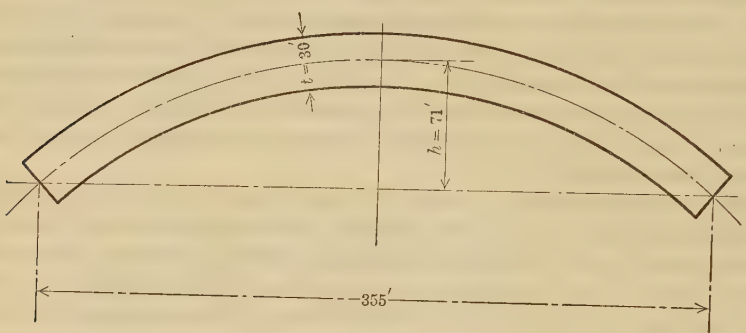

FIG. 6.

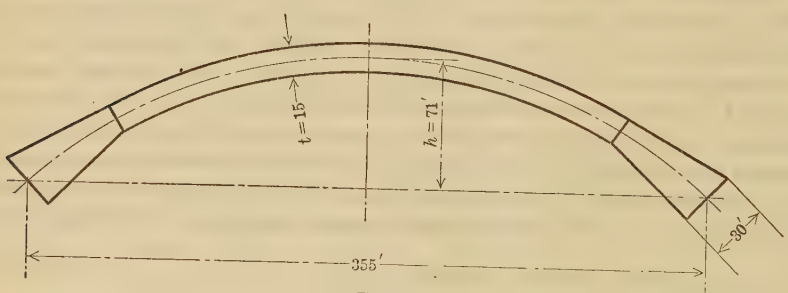

FIG. 7.

sound part of the beam will remain in the shape of a new arch which can act as a real arch, and the size of this new arch may be determined by the theory of least work. It might be objected that such cracks if they really existed would have been reported on dams already built. Calculations, however, show

that the sum of the width of all the cracks may be only $\frac{1}{7\,000}$ in., hence they would be considered to be hair-cracks only, if found at all. This action of the thick flat arch has been only vaguely understood and denominated as "wedge" action, although a wedge is not a recognized structural member.

It is clear that the new arch will show much higher stresses than those calculated on the whole section by the ordinary cylinder theory. The author, as well as F. A. Noetzli, Assoc. M. Am. Soc. C. E., believes that the stresses can be found from deflection measurements of the arch. The writer thinks that this method would lead to erroneous views. The modulus of elasticity is a greatly varying quantity. Even in a common cylinder test, the moduli on different sides of the cylinder often vary more than 100% and the figures given by authors are only averages. The best information on this subject is found in the report of the Committee of the Austrian Engineering Society on tests on a number of masonry and reinforced concrete arches (made in 1892 at an expense of $20 000) and the scientific treatise by Mr. J. A. Spitzer on the reinforced concrete arches of these tests.* Arches were loaded with known weights, acting at known points, and the deflections and changes of angles were observed carefully at many points. From the deflection, the modulus of elasticity was calculated to be 2 000 000 lb. per sq. in., up to a load about 30% less than that producing the first visible crack. At the first crack, the modulus was about one-half the value mentioned, and for higher loads was still much less.

There is another uncertainty about the modulus of elasticity, due to the flow or plasticity of concrete, and many phenomena cannot be explained unless a low modulus is assumed. Such is the case with the effect of a change in temperature or the shrinkage of the concrete, which are slow actions, in which the time factor plays an important rôle. Recent tests on the shrinkage of concrete and on the effect of temperature lead to the conclusion that the modulus of elasticity lies between 500 000 to 1 000 000 lb. per sq. in. for these actions and, in some cases, drops to 100 000 lb. per sq. in. and less. The stresses cannot be found from the deflections in an arch with contraction joints or cracks (unless the arch is properly reinforced), because the distribution of stresses in those planes is widely different from that obtained by the straight-line theory, once tensile stresses are calculated in the extreme fibers.

In this connection the writer wishes again to call the attention of the Profession to the unscientific character of the plain masonry or concrete arch dam, as commonly built. Fig. 6 shows a horizontal section of an arch dam which has been constructed. A non-reinforced arch like this might have been designed by the old Roman engineers. A modern bridge designer would never use it, because the stresses from temperature and shrinkage action will be larger than those from the live loads, and such an arch will crack. The latest information available indicates that the greatest shortening due to the combination of a fall in temperature and shrinkage of concrete for an arch of that thickness is about $\frac{1}{1\,800}$ of the length. The pull or negative thrust

* *Zeitschrift*, Osterreichischer Ingenieur und Architecten Verein, No. 20 (1896).

caused by this shortening, when the modulus of elasticity is 1 000 000 lb. per sq. in., can be obtained for an arch of that particular relation of rise to span, by the formula:

$$H = \frac{66\,000\,t}{\left(\frac{h}{t}\right)^2 + 1.26} \text{ lb. per lin. ft.}$$

when h equals the rise of the arch and t the thickness, in feet. This formula has been obtained in a similar manner, as shown in the writer's discussion of the Huacal Dam, previously referred to, except that the effect of shear was considered. By substituting $t = 30$ ft., $h = 71$ ft., $H = 9\,550 \times t = 286\,500$ lb. per lin. ft., and using Mr. Noetzli's approximation of $\frac{2}{3} \times h$ as the leverage of this pull on a section at the abutment, the moment there becomes:

$$286\,500 \times 47.33 = 13.55 \times 10^6 \text{ ft-lb.}$$

The moment of resistance of the section being $\frac{30^2}{6}$, the extreme fiber stresses at the abutment are $\pm$ 90 500 lb. per sq. ft. The component of the pull, tangent to the arch at the abutment, is about 220 000 lb. and the corresponding tension, $\frac{220\,000}{30} = 7\,333$ lb., or the extreme fiber stresses, — 97 833 lb. and + 83 167 lb., respectively. The arch will crack or expansion joints will open, which fact is amply confirmed by actual observation of that dam. The water pressure does not entirely relieve the arch of the tensile stresses. It produces only a compression of 47 000 lb. per sq. ft.; therefore, the arch must act as a reduced section, as previously described for the very thick arch, with much higher stresses than the 47 000 lb. assumed by the designer.

Assume, now, that the arch has a section similar to that shown in Fig. 7. On account of the enlargement at the abutment, the negative thrust will be comparatively larger than that for an arch of a uniform thickness, and approximately is given by:

$$H = \frac{100\,000\,t}{\left(\frac{h}{t}\right)^2 + 2}$$

or, $H = 61\,500$ lb.

The leverage of this force about a section at the abutment also becomes greater and will be assumed to be 50 ft.; therefore, the moment at the abutment is 61 500 $\times$ 50 = 3 075 000 lb., and the extreme fiber stresses are $\frac{3\,075\,000}{\frac{30^2}{6}} = 20\,500$ lb. per sq. ft. The direct tensile stresses are, approximately, 1 440 lb., and the total stresses are — 21 940 and + 18 060 lb., respectively. The compressive stresses from the water pressure being 47 000 lb. at the abutment, will more than counterbalance the tensile stresses, but will not prevent the opening of contraction joints when the dam is empty. This can be prevented only by properly reinforcing the arch and

anchoring the reinforcing bars into the rock walls at the abutments. The writer hopes to have shown that with a saving of nearly 50% in masonry, lower stresses are produced in a reinforced concrete dam than in the plain concrete arch of to-day, and this without the fear of dangerous cracks.

That reinforced concrete arches are much more economical than plain concrete arches was unquestionably brought out by the Austrian tests, previously mentioned. A concrete arch of 76 ft. span, 15 ft. rise, 6.5 ft. wide, and 27.5 in. thick throughout, was tested to failure by the application of a total load of 185 000 lb., uniformly distributed on one side of the arch.

A reinforced concrete arch of the same span, rise, width, with a crown thickness of only 13¾ in., increasing to 25 in. at the abutment, failed at a much higher load, namely, 322 000 lb., distributed on the half span.

The arch dam of the future should not show larger cracks than modern arch bridges, and with richer concrete and proper reinforcement, there is no reason why stresses as high as those calculated for other reinforced concrete work should not be used. Thoughtful engineers may object to the use of reinforcement on account of the danger of the steel rusting by the action of intrusive water; there is no more danger that this will happen in dams than in reinforced concrete water tanks and reinforced concrete sewers, the general use of which is older than modern reinforced concrete buildings and bridges. Of course, richer concrete must be used, and the face of the dam should be water-proofed by any of the excellent methods which have proved successful in tunnels and high tanks. It is a fact, that most reinforced concrete tanks, in which the concrete acts in tension only, are comparatively much thinner than most concrete arch dams. Should similar structures in which the concrete acts primarily in compression be made thicker for the only reason that there may be some similarity with a gravity dam?

According to the writer's opinion, every arch ring of a dam should be designed to carry the entire water pressure, including all secondary stresses, proper allowance should be made for the effect of shrinkage and temperature in thick sections being very much less than in thin sections, as the concrete in the interior has no chance to dry out or to become affected to any extent by a change in temperature, and cracks should be prevented by proper reinforcement.

In large arch dams, as a rule, it is uneconomical to assume the arch rings to be affected only by the water pressure. If the weight of the arch ring is also considered, it will be found that the most economical design is obtained by inclined arches, the inclination being given by the ratio of water pressure per square foot to total weight of the inclined ring per linear foot. The reinforced concrete arch dam should be considered as a monolith, affected by the entire water pressure, the weight of the structure, and the inclined reactions of the side-walls. The inclined rings will take up most directly the inclined reactions and will relieve the base of the dam from the superimposed load; therefore, the supposed favorable influence of Poisson's ratio cannot be considered when the reservoir is full and the high stresses from the cantilever action do not exist at the base in a reinforced concrete dam. The rings have different inclinations at different depths and become nearly horizontal at the base of the dam.

During a stay on the Pacific Coast, the writer made a design on these lines for a reinforced concrete arch dam 210 ft. high, and 400 ft. long at the river bed, and found that a section 35 ft. thick at the base had much smaller stresses than would be found in a plain concrete curved dam of twice the thickness.

Engineers may be afraid to use comparatively thin, reinforced concrete arches on account of the supposed column action in such slender arch rings. The writer knows of only one test of a concrete arch where failure was produced by an uniform load over the entire span. This test was made by the Blaubeuren cement mill, in Ehingen on the Danube, on a three-hinged arch of 66 ft. span, 5 ft. rise, $4\frac{3}{4}$ in. thick at the crown, $8\frac{3}{4}$ in. thick at the abutment, and $12\frac{1}{2}$ in. thick at the $\frac{1}{4}$-in. point. The structure was loaded for 7 years from 1896 to 1903, and failed after the last increase of load had been on it for 4 years. The maximum compressive stresses were calculated to be 840 000 lb. per sq. ft., although 12-in. cubes, cut from the structure afterward, failed at 954 000 lb. A reduction of compressive stress is found in tests of short columns, and the test of this slender arch does not indicate that column action is to be feared in arched dams.

The writer hopes that Professor Cain's paper will be considered carefully by all engineers interested in dam construction and will induce them to use more science and less speculation in the design of single and multiple-arch dams.

B. F. Jakobsen,* Assoc. M. Am. Soc. C. E. (by letter).—The author has presented an interesting and valuable paper on an important subject. He seems to possess in an unusual degree the happy faculty of the born mathematician of presenting a complicated and highly mathematical subject in such a manner as to make it readable and, therefore, doubly valuable. Even a cursory reading of the paper affords an understanding of the assumptions made therein, the analytical methods used, and the methods given for using the formulas.

Equations (12) and (13) are general formulas and, therefore, should apply for any value of the radius, r. As a matter of fact, when r approaches infinity, and the central angle approaches zero, the two formulas do give the correct bending moments for a straight beam fixed at both ends and loaded uniformly with p lb. per sq. ft. From Equations (12) and (10), we have, writing x in place of ϕ_1:

$$M_0 = \frac{x - \sin(x)}{x} \cdot \frac{p\, k^2\, 2\, x \sin(x)}{\left(1 + \frac{k^2}{r^2}\right)[x^2 + 0.5\, x \sin(2x)] - 2 \sin^2(x)}$$

If the width of the canyon is L ft., then:

$$r = \frac{L}{2 \sin(x)} \qquad (20)$$

since x is half the central angle. Also,

$$\sin(x) = x - \frac{x^3}{6} + \frac{x^5}{120} - \frac{x^7}{5\,040} + \cdots$$

and for small values of the angle, x,

$$\sin(x) = x - \frac{x^3}{6} \qquad (21)$$

* Designing Engr., San Joaquin Light and Power Corporation, Fresno, Calif.

For an angle of 60°, the error in Equation (21) is less than 1 per cent. Introducing Equations (20) and (21) in the equation for M_0 and dividing by x^4:

$$M_0 = \frac{2\,p\,k^2}{6} \frac{1 - \frac{x^2}{6}}{-\frac{x^2}{18} + 4\,\frac{k^2}{L^2}\left(2 - \frac{4}{3}x^2 + \frac{5}{18}x^4 - \frac{x^6}{54}\right)} \quad \ldots\ldots(22)$$

and for $x = 0$,

$$M_0 = \frac{p\,L^2}{24}$$

and this is the equation for the bending moment in the center of a straight beam fixed at the ends and loaded uniformly with p lb. per sq. ft.

Likewise, Equation (13) for the moment at the abutments becomes, after introducing Equations (20) and (21) and dividing by x^4:

$$M = \frac{2\,p\,k^2}{6} \frac{2 - \frac{7}{12}x^2 + \frac{x^4}{24}}{-\frac{x^2}{18} + 4\,\frac{k^2}{L^2}\left(2 - \frac{4}{3}x^2 + \frac{5}{18}x^4 - \frac{x^6}{54}\right)} \quad \ldots\ldots(23)$$

and for $x = 0$,

$$M = \frac{p\,L^2}{12},$$

which is the formula for the bending moment at the end of a straight beam loaded uniformly with p lb. per sq. ft.

Equations (22) and (23) may be used for computing the moments for any angle likely to occur in dam design. As already pointed out, Equation (21) is correct within 1% for central angles of 120 degrees. For small values of the central angle, Equations (22) and (23) give nearly exact values.

From Equations (22) and (23), the relative values of the bending moments can be more readily seen than from the original equations. It is evident that the bending moment at the abutment is always considerably larger than that at the crown, but as x increases this difference decreases. The statement* by F. A. Noetzli, Assoc. M. Am. Soc. C. E., in his paper on "The Relation Between Deflections and Stresses in Arch Dams", that "in a fixed arch, the bending moments at the arch abutments resulting from the thrust, H, are approximately twice as large as at the crown," is seen to be incorrect; for example, for a central angle of about 120°, that is, x is nearly one, the bending moment at the abutment is less than 1.5 times the moment at the crown, and for larger values of the central angle this is further reduced. It may also be noted, that Equation (14) for the shear may be obtained from Equation (13) by differentiation, as is the case for a straight beam.

Equation (15) which gives the deflection at the crown for an arch fixed at the abutments, was also checked by substituting Equations (20) and (21), dividing by x^6, and setting $x = 0$, when Equation (15) becomes:

$$\text{Deflection} = \frac{p\,L^4}{16\,E\,t^3}$$

which is the deflection in the center of a straight beam fixed at the ends and loaded uniformly with p lb. per sq. ft.

* See p. 289.

No doubt the same result can be found from the formulas given for the hinged arch; the writer does not believe that the assumption of a hinged arch is ever satisfied in dams, although the assumption of fixed ends certainly is, whenever no excessive tension is found at the abutments by the formulas for fixed arches.

The author assumes a constant modulus of elasticity and neglects the deformation due to shear and the influence of the water soaking of the upstream face. In a thin arch, this should not lead to any appreciable error and, in the writer's opinion, the long, slender arches near the top of a dam are those most difficult to design properly, since little assistance is afforded from cantilever action. When no tensile stresses are found at the abutments, and when the central angle is large, say, 120° or thereabouts, the spreading of the foundation must be negligible, especially as the total force on the abutment is small for a thin arch, and it seems to the writer that, in this case, the assumption of fixed ends agrees very closely with the actual conditions. With the nearly exact formulas of the author, it is possible to determine the stresses in the top of the dam instead of relying on the Rankine formula and using low stresses in the computations in order to take care of the uneven distribution of stresses through the section.

To facilitate calculations, the writer has plotted the author's Table 2 on Fig. 8. This was done by plotting on semi-logarithmic paper the values of $\frac{2 \sin \phi_1}{D_0}$ for the various angles, one curve for each value of $\frac{t}{r}$, and then interpolating. To illustrate the formulas with a thin arch, suppose $\frac{t}{r} = 0.02$ and $2\phi_1 = 40°$, and let $p = 864$ lb. per sq. ft., $r = 50$ ft. and, therefore, $t = 1$ ft. The Rankine formula gives:

$$r' \times p' = s \times t = r \times p$$

$$s = \frac{864 \times 50}{1} = 43\,000 \text{ lb. per sq. ft.} = 300 \text{ lb. per sq. in.}$$

The author's formulas give:

$$p\,r = 864 \times 50 = 43\,200 \text{ lb. per sq. ft.}$$

$$p\,r - P_0 = 2\,750 \times \frac{864}{12 \times 50} = 3\,980 \text{ lb. per sq. ft.}$$

At the crown:

Extrados = 440 lb. per sq. in. (compression)
Intrados = 108 lb. per sq. in. (compression)

At the abutment:

Extrados = 57 lb. per sq. in. (tension)
Intrados = 607 lb. per sq. in. (compression)

For an arch with the same thickness and radius and the same loading, except that $2\phi_1 = 180°$,

$$p\,r - P_0 = 6.7 \times \frac{864}{600} = 14.4 \text{ lb. per sq. ft.}$$

Therefore, practically, $pr = P_0$, the bending moments are practically zero, and the Rankine formula applies very nearly.

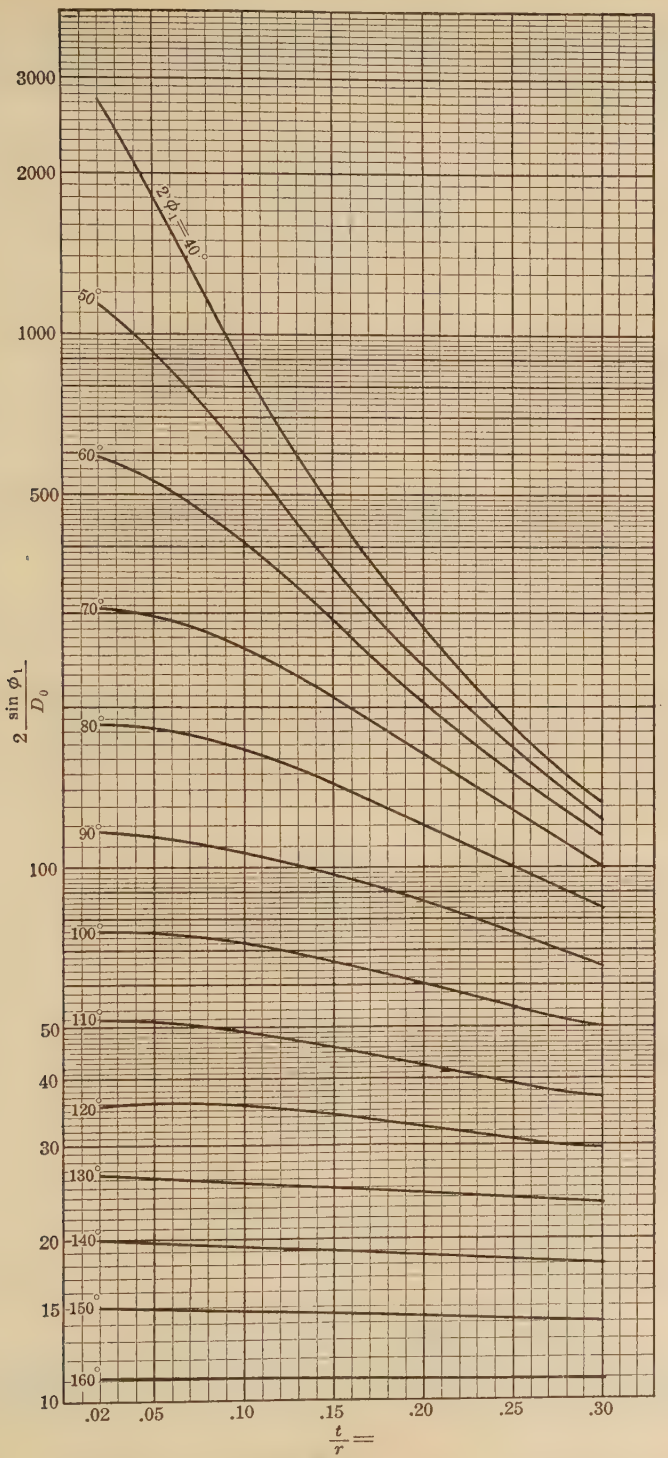
3000
2000
1000
500
100
50
40
30
20
15
10
$2\frac{\sin\phi_1}{D_0}$
$2\phi_1 = 40°$
50°
60°
70°
80°
90°
100°
110°
120°
130°
140°
150°
160°
.02
.05
.10
.15
.20
.25
.30
$\frac{t}{r} =$

FIG. 8.

As an example from an actual dam, we may consider the Kerckhoff Dam.* This is a constant-angle arch dam, approximately 110 ft. high, with Taintor gates, 14 ft. high, on top. At Elevation 970, with water standing 10 ft. above the top of the Taintor gates:

$$h = 25 \text{ ft.};$$
$$t = 9.88 \text{ ft.};$$
$$r' = 205 \text{ ft.};$$
$$r = 200 \text{ ft.};$$
$$\frac{t}{r} = 0.0494$$
$$2\phi_1 = 122°$$

The author's formulas for an arch fixed at the abutments give:

$$p \times r = 321\,000 \text{ lb.}$$
$$p \times r - P_0 = 2\,220 \text{ lb.}$$

Therefore, at the crown:

Extrados = 257 lb. per sq. in. (compression)
Intrados = 191 lb. per sq. in. (compression)

and, at the abutments:

Extrados = 156.5 lb. per sq. in. (compression)
Intrados = 300 lb. per sq. in. (compression)

The Rankine formula gives 225 lb. per sq. in. The maximum shear exists at the abutment; by the author's Equation (14), and assuming a parabolic distribution through the cross-section, it is only 2 lb. per sq. in., and the deformation due to shear, it would seem, would certainly be negligible.

At Elevation 900:

$$h = 95 \text{ ft.};$$
$$t = 289 \text{ ft.};$$
$$r' = 157.7 \text{ ft.};$$
$$r = 143.3 \text{ ft.};$$
$$\frac{t}{r} = 0.201;$$
$$2\phi_1 = 82° \ 40'.$$

This gives:

$$p \times r = 935\,000 \text{ lb.}$$
$$p \times r - P_0 = 350\,000 \text{ lb.}$$

Therefore, at the crown:

Extrados = 357 lb. per sq. in. (compression)
Intrados = 75 lb. per sq. in. (tension)

At the abutments:

Extrados = 246 lb. per sq. in. (tension)
Intrados = 568 lb. per sq. in. (compression)

* See the writer's article, "Design and Construction Features of the Kerckhoff Dam", *Engineering and Contracting*, June 8th, 1921, p. 565.

The Rankine formula gives 225 lb. per sq. in. Computing the shear at the abutment as before, we find the maximum shear equals 83.3 lb. per sq. in. Although the maximum compression has about doubled, the shear, in this case, is forty times as great as previously stated.

It is worth noting in this connection that the water load gives moments of opposite signs to the moments due to a rise in temperature (see Figs. 2 and 4).

A temperature increase of 5° Fahr., for the Kerckhoff Dam, at Elevation 900, and with $E = 144 \times 3\,000\,000$; $e = 0.0000055$, gives:

$$H_0 = 128\,000 \text{ lb.} = 30.7 \text{ lb. per sq. in.}$$

and, since for any angle, ϕ,

$$H = H_0 \cos \phi$$

there is obtained for the temperature stresses alone:

At the crown:

Extrados = 47.3 lb. per sq. in. (tension)
Intrados = 108.7 lb. per sq. in. (compression)

At the abutments:

Extrados = 172 lb. per sq. in. (compression)
Intrados = 126 lb. per sq. in. (tension)

Combining the water loading due to 95 ft. of head with the loading due to a temperature rise of 5° Fahr.:

At the crown:

Extrados = 310 lb. per sq. in. (compression)
Intrados = 34 lb. per sq. in. (compression)

At the abutments:

Extrados = 74 lb. per sq. in. (tension)
Intrados = 442 lb. per sq. in. (compression)

It is quite evident, therefore, that grouting the dam under pressure or closing it at or near the end of the coldest season, as suggested by L. R. Jorgensen,* M. Am. Soc. C. E., must be beneficial. A comparatively large increase in the compressive stresses is not a matter of great importance, and the method evidently makes it possible to reduce the undesirable tensile stresses.

That the influence of shear will be much greater with thick arches than with thin arches with the same central angle may be seen by comparing the shear in the two examples given by the author on page 239.

In the first example:

$$p\,r - P_0 = 0.00265\,p\,r$$

Since $t = 4$ ft.,

$$P = 0.25\,p\,r$$

in which P is in pounds per square foot, and if parabolic distribution of the shear is assumed, then,

$$S = \frac{1.5 \times 0.00265\,p\,r \times 0.866}{4} = 0.00086\,p\,r$$

* "Improving Arch Action in Arch Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

in which S is in pounds per square foot, and, therefore,

$$\frac{P}{S} = 290$$

Therefore, the shear is certainly negligible compared with the normal stresses.

From the second example, we have:

$$p\,r - P_0 = 0.219\,p\,r$$

and, since $t = 40$ ft.,

$$P = 0.0223\,p\,r$$

in which P is in pounds per square foot, and the shear is,

$$S = 0.0071\,p\,r$$

in which S is in pounds per square foot, and, therefore,

$$\frac{P}{S} = 3.14.$$

In the first example, the maximum shear is about one-third of 1% of the normal stress, while in the latter case, the shear is about one-third of the normal stress, or relatively speaking, nearly one hundred times greater than in the first example.

If r is assumed to be the same in both cases, but p in the second case is ten times as great as in the first case, because t is ten times greater, then the maximum shear in the two cases is:

$$\frac{0.00086}{0.071} = \frac{1}{82}$$

In the second case, the shear stresses are eighty-two times greater than in the first case.

It would seem, therefore, that for thick arches, when the central angle is not too small, the author's Equation (5) should include the elastic work for shear. This is:*

$$L = \frac{1}{2\,G}\int d\,x \int\int S^2\,dy\,dz$$

or, for a rectangular section of unit width, since,

$$G = \frac{E}{2.4}, \text{ and } d\,x = r\,d\,\phi$$

$$L = 2.88\,\frac{r}{2}\int_0^{\phi_1} \frac{S^2\,d\,\phi}{E\,t}$$

Also, in the thick arch, there is more likelihood of a considerable difference in temperature between the up-stream and down-stream faces, and the assumption of a constant modulus of elasticity is much more questionable, because the difference between the stresses in the up-stream and down-stream faces is considerable and much greater than is the case with thin arches. Finally, in actual dams, the thick arches are near the base and considerably affected by cantilever action and the weight of the concrete above.

In the example of the Kerckhoff Dam, the tension in the up-stream face at the abutment, at Elevation 900, would be 246 lb. per sq. in., if it is assumed that

* "Elasticität und Festigkeit", C. Bach, Third Edition, p. 349.

the arch sustains the entire water pressure. The arch would fail, therefore, if it was loaded as assumed; but if account is taken of the shear deformation and of the variability of the modulus of elasticity,* a different result would be obtained, and this arch would probably require a load several times greater than the assumed load before it would fail.

The author's Equation (15) gives the deformation at the crown for an arch fixed at the abutment. From this formula, we should be able to estimate the deflection of a free arch. For thin arches, the formula should give correct values, that is, values which agree with the modulus of elasticity of the material in the arch. For thick arches, the writer believes the correspondence would be much smaller for the reasons previously given. As an illustration of very large deformations in a concrete dam which has not produced cracks, the Salmon Creek Dam† might be mentioned. At Elevation 1075, there is a very decided knee in the deflection curve, and this would require a vertical tensile stress of 2 750 lb. per sq. in., if the deformation is to be accounted for by deflection due to bending under the influence of a uniformly distributed load. In spite of this, no cracks have been reported in the down-stream face, and it is not likely, therefore, that any exists. The assumption generally made, that the deflection is due to bending alone, is not correct.‡ The deflection due to shear is likely to be several times greater than the deflection due to bending, so that the tensile stress, when correctly computed, would be, perhaps, one-third of the value previously given. If this stress really existed, it would not fail to produce cracks, and, therefore, it may serve as an excellent illustration of the great circumspection necessary when concluding, from the measured deflections, the magnitude and nature of the stresses in a large concrete arch.

It should also be borne in mind that the deflection which occurs the first time the dam is loaded is not an elastic deformation, or as expressed by Messrs. Eddy and Turner§:

"Initial loading, however, is certain to develop noticeable deviations from perfect elasticity because of shrinkage stresses developed in the hardening of the cast concrete. A few repetitions of loading largely eliminates such deviations, so that the theory of work and the elastic theory of deformation becomes applicable and proves a practical and precise method of treatment."

To this should be added the influence of the internal heating due to the setting or curing of the cement and also the influence of lateral stresses which in a large concrete dam cannot be altogether negligible.

The author has solved an important problem in dam design in an exact and sufficient manner, and his solution will be of great help to engineers who have to design large arch dams.

FRED A. NOETZLI,|| ASSOC. M. AM. SOC. C. E. (by letter).—Professor Cain's paper is a valuable contribution to the theory of arch dams, and it is to be hoped

* See the writer's discussion on "Gravity and Arch Action in Curved Dams", by F. A. Noetzli, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 95.

† "The Relation Between Deflections and Stresses in Arch Dams", by F. A. Noetzli, Assoc. M. Am. Soc. C. E., Fig. 2, p. 297.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 99, especially Fig. 32.

§ "Concrete-Steel Construction", Part I, p. 76.

|| Chf. Engr., Beckman and Linden Eng. Corporation, San Francisco, Calif.

that tests will soon be made in order to determine to what extent the behavior of such dams agrees with exact theory.

The study of the problem of arch dams, and with due consideration of the historical development of arch dam designs, has brought the writer to the conclusion that in the present state of knowledge with regard to modulus of elasticity, range of temperature, etc., of the dam material, it would be just as well to use, as a beginning, approximations for the sake of simplicity. More or less successful attempts toward a correct solution have been made. Most of these calculations, however, had their merits buried, for the average reader, in higher mathematics, etc., and the new formulas, due to their complicated nature, were ignored by the busy designer.

The assumptions on which, at the present time, any theory of arch dams has to be based, still involve a considerable degree of uncertainty. This is, in the case of large arch dams, particularly true with regard to the modulus of elasticity, range of temperature, degree of shrinkage of concrete, "fixity" of the arches, swelling, lateral expansion (Poisson's ratio), secondary arch action, "wedge" action, etc., so that a high degree of accuracy in the formulas used is not of much value. Before extensive tests have been made, the modulus of elasticity can hardly be judged and its uniformity guaranteed within 20% of any assumption, and this is also true of the range of temperature, as well as the degree of shrinkage. Assuming for the other uncertainties involved another 20%, for all four assumptions together there is a probable error of,

$$\sqrt{20^2 + 20^2 + 20^2 + 20^2} = 40\%$$

which is probably conservative for thick dams. Consequently, if approximate formulas are within 20 to 25% of the theoretically correct values they may be considered, for the present, as sufficiently accurate as compared to the probable error of 40% in the assumptions.

Theoretically exact formulas, however, are desirable as long as they can be expressed in a simple form. Fortunately, the author's final equations go far toward this end, which makes his paper doubly valuable.

Professor Cain has also included in his investigation the thick and the flat arches, as they may occur in the lower parts of arch dams. Mr. Mensch has expressed doubts as to the correctness of some of the deflection formulas and certain coefficients of Table 1. There is no question that by also considering the effect of shear, slightly larger deflections will be found for short and thick arches than are obtained from the author's Equation (15).

However, it is rather questionable whether such thick and flat structures should be classified as arches. For instance, the author has calculated that for an arch with a radius of 135 ft., with a thickness of 40 ft., and a central angle of 40°, only 2½% of the load is supported by what is usually called arch action. The remainder of the load, or 97½%, is assumed, by the author, to be carried by beam action.

Mr. Mensch has shown why beam action cannot be relied on to carry the load. The writer wishes to point out why "secondary" arch action within the masonry may prevail in such a case. From Fig. 5, it is seen that there is the possibility that within the large arch, a secondary arch is formed,

approximately as shown by the dotted lines.* For the size of arch chosen by the author, a smaller arch with a radius of approximately 52 ft. and a thickness of 20 ft., may be inscribed. According to the cylinder formula, the stress in the large arch would be:

$$f_c = \frac{p\,r}{t} = p\,\frac{135}{40} = 3.37\,p$$

The stress in the small arch for the same load would be obtained at,

$$f'_c = \frac{p\,r}{\frac{t}{2}} = p\,\frac{52}{20} = 2.60\,p,$$

or even less than in the large arch. Thus, the small arch, due to the shorter radius, seems to be stronger than the large one. This reasoning indicates that secondary arch action will prevail in such a case. The size, shape, and stresses of the secondary arch can be determined by Castigliano's theory of least work. For the present purpose, it will be sufficient to indicate that arch formulas applied to such "arches" as shown in Fig. 5 may lead to erroneous conclusions. Giving due consideration to such limitations of the author's theory, it is seen that the discrepancies between his exact arch deflection formula and the writer's approximation† is comparatively small for all true arches as may occur in arch dams.

It is of further interest to note that the author's method of calculating the arch deflection stresses, due to rib-shortening and temperature changes, furnishes results that check almost exactly those obtained by the writer's method, using the coefficients from the curves of Figs. 4 and 5 of his paper, "The Relation Between Deflections and Stresses in Arch Dams".‡ Consequently, the arch deflection stresses, calculated from measured deflections, will also be about the same if the same assumptions as to temperature, etc., are made.

It may not be amiss to warn against an indiscriminate application of the author's as well as the writer's "deflection stress" formulas. Those equations give reliable results for unreinforced arches only if, during the state of the deformation considered, the arches have no open tension cracks. If the calculations show tension in a dam to such a degree that the construction joints may open (for instance, at a tension of 50 lb. per sq. in.), these formulas can no more be applied directly. They then will indicate only that the compression stresses are distributed unevenly over the arch.

Mr. Jakobsen questions the correctness of a statement made by the writer on page 289 of his paper previously mentioned, that "in a fixed arch, the bending moments at the arch abutments resulting from the thrust, H, are approximately twice as large as at the crown." This is no doubt due to a misunderstanding on the part of Mr. Jakobsen, inasmuch as the thrust, H, goes through the center of gravity of the arch, and this one is located, for arches to a central angle of 120°, at very nearly one-third of the rise from the crown and two-thirds of the rise from the abutments of the arch. Consequently, the lever arm for H, at the abutments, is twice as large as at

* On this point, see, also, the writer's investigations on "vertical arching" in dams, *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 36.

† "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 11.

‡ See pp. 304 and 305.

the crown and, therefore, the bending moments also, due to H. Mr. Jakobsen apparently mistakes the moment given by the Equations (21) and (22), which include thrust and bending, with the moment from H (Fig. 4), which is due to temperature or rib-shortening.

William Cain,* M. Am. Soc. C. E. (by letter).—To assist the practical engineer in the solution of "the circular arch under normal loads", Tables 1 and 2 have been much extended, and are given as Tables 4, 5, and 6. The coefficients in the starred rows were computed, the others being interpolated. In effecting the interpolation, curves were drawn for $\frac{t}{r}$ constant, and $2\phi_1$ variable, so that separate curves were drawn for $\frac{t}{r} = 0.00,\ 0.02,\ \ldots,\ 0.30$, in turn, from which the ordinates could be obtained for given values of $2\phi_1$. Where the curvature was appreciable, the results, for the interpolated values, are approximate, since they depend on how the curves are drawn between the points, as given by computation; but it is believed that the interpolated coefficients are sufficiently near the truth for use in practice. In fact, most of the interpolated values are practically exact. The regularity of the curves drawn to a large scale is the most convincing proof of the accuracy of the computations, although the formulas were changed to other but equivalent forms, for a few independent computations, as an additional check.

TABLE 4.—Circular Arch Fixed at Ends, Under Normal Loads.

Values of coefficient, c, in $\eta = c\left(\frac{p\,r^2}{E\,t}\right) = c\left(\frac{p'\,r\,r'}{E\,t}\right)$.

$2\phi_1$	$\frac{t}{r}=0$	$\frac{t}{r}=0.02$	$\frac{t}{r}=0.06$	$\frac{t}{r}=0.10$	$\frac{t}{r}=0.15$	$\frac{t}{r}=0.20$	$\frac{t}{r}=0.25$	$\frac{t}{r}=0.30$
*40°	1.877	1.708	0.994	0.542	0.287	0.173	0.115	0.081
45°	1.877	1.765	1.190	0.730	0.430	0.260	0.180	0.130
*50°	1.878	1.806	1.382	0.941	0.580	0.377	0.261	0.190
55°	1.879	1.825	1.510	1.120	0.750	0.515	0.365	0.270
*60°	1.881	1.845	1.606	1.277	0.911	0.651	0.477	0.360
65°	1.881	1.857	1.670	1.400	1.065	0.800	0.610	0.470
*70°	1.882	1.863	1.726	1.505	1.204	0.942	0.736	0.581
75°	1.883	1.867	1.760	1.580	1.325	1.070	0.865	0.700
*80°	1.884	1.873	1.791	1.648	1.425	1.198	0.995	0.825
85°	1.885	1.876	1.810	1.695	1.510	1.310	1.110	0.950
*90°	1.886	1.879	1.828	1.735	1.577	1.400	1.223	1.060
95°	1.887	1.881	1.840	1.765	1.630	1.480	1.320	1.160
*100°	1.889	1.884	1.851	1.789	1.679	1.546	1.404	1.262
110°	1.892	1.890	1.860	1.820	1.745	1.645	1.540	1.420
*120°	1.894	1.894	1.878	1.848	1.794	1.723	1.640	1.549
130°	1.897	1.897	1.885	1.865	1.830	1.770	1.710	1.640
*140°	1.901	1.900	1.893	1.878	1.850	1.812	1.766	1.713
160°	1.910	1.909	1.905	1.895	1.880	1.860	1.830	1.800
*180°	1.918	1.918	1.916	1.911	1.903	1.891	1.877	1.859

Note.—The starred rows were computed; the other rows were interpolated.

The curves for the coefficients of Table 4, for an arch "fixed at the ends", and for various values of $2\phi_1$ $\left(\frac{t}{r}\text{ varying}\right)$, are shown in Fig. 9.

On account of the small scale used, the interpolated values are not shown, especially as Table 4 will be generally consulted for any required coefficient.

* Prof. of Math., Univ. of North Carolina, Chapel Hill, N. C.

Table 5 gives the deflection coefficients for an arch "hinged at the ends". No interpolated values are given, as, generally, it was thought that the results would only apply to the thinner arches.

TABLE 5.—CIRCULAR ARCH HINGED AT ENDS, UNDER NORMAL LOADS.

Values of coefficient, c, in $\eta = c\left(\frac{p\ r^2}{E\ t}\right) = c\left(\frac{p'\ r\ r'}{E\ t}\right)$

$2\phi_1$	$\frac{t}{r}=0$	$\frac{t}{r}=0.02$	$\frac{t}{r}=0.06$	$\frac{t}{r}=0.10$	$\frac{t}{r}=0.15$	$\frac{t}{r}=0.20$	$\frac{t}{r}=0.25$	$\frac{t}{r}=0.30$
*40°	1.566	1.541	1 363	1.108	0.811	0.590	0.437	0.332
*50°	1.568	1.557	1.478	1.342	1.138	0.938	0.773	0.625
*60°	1.570	1.565	1.518	1.453	1.331	1.190	1.047	0.914
*70°	1 573	1.570	1.549	1.509	1.436	1.344	1.243	1.138
*80°	1.576	1.574	1.562	1.538	1.494	1.436	1.367	1.292
*90°	1.579	1.578	1.568	1.556	1.528	1.490	1.444	1.392
*100°	1.584	1.583	1.578	1.569	1.550	1.525	1.494	1.458
*120°	1.593	1.593	1.590	1.586	1.578	1.566	1.551	1.533
*140°	1.605	1.605	1.604	1.602	1.597	1.592	1.584	1.574
*180°	1.637	1.637	1.636	1.636	1.634	1.632	1.630	1.627

NOTE.—All values were computed.

Table 6 gives the values of $\frac{2 \sin \phi_1}{D_0}$, to be used in the computation of either $(p\ r - P_0)$ or H (for temperature changes), for an arch "fixed at the ends", or one without hinges.

On account of the great variation in the coefficients, Mr. Jakobsen's plan of using semi-logarithmic paper for the complete graph, was adopted. It is shown in Fig. 10.*

In his paper entitled, "The Relation Between Deflections and Stresses in Arch Dams", page 305, Mr. Noetzli has derived the formula:

$$H_t = k_t \frac{t^3}{h^2} E\ e\ t_0$$

for the thrust on the crown section, due to a temperature change of t_0 degrees, in which the influences of moment and tangential thrusts are considered. The coefficients, k_t, are to be taken from Fig. 5 of his paper.

The writer's formula for H, the thrust due to a temperature change, for an arch without hinges, can be expressed in the form last given, where k_t is replaced by,

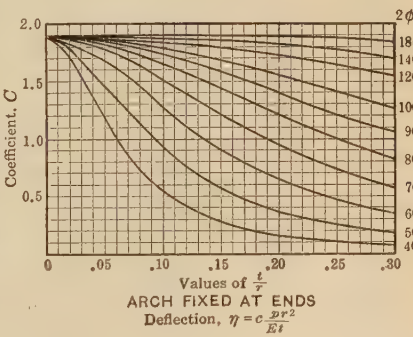

FIG. 9.

* To include properly the interpolated values, Figs. 9 and 10 should be drawn to larger scales. A very satisfactory size for the engraved part of the semi-logarithmic paper is 12 in. square, with two logarithmic scales in the vertical direction. If the upper limit is marked 1 000, all values of Table 6 over 1 000, will not be represented; but such values are rarely needed, since they correspond to the smaller values of $\frac{t}{r}$ coupled with the smaller central angles.

$$\frac{(1 - \cos \phi_1)^2}{12} \cdot \frac{2 \sin \phi_1}{D_0},$$

so that numerical values are readily computed by utilizing the values given in Table 6. This was done for a wide range of values of $\frac{t}{h}$ and $2\phi_1$. The numerical results were compared with the corresponding ones taken from Mr. Noetzli's Fig. 5 and were found to agree very well, thus affording a valuable check on the accuracy of the computations. Mr. Noetzli's Fig. 5 would have to be extended to cover the same range as the writer's Table 6 or Fig. 10; but, even then, it is compact and avoids the use of semi-logarithmic paper, so that there seems to be a practical advantage in expressing results in terms of $\frac{t}{h}$. A corresponding table should preferably give k_t to three significant figures.

TABLE 6.

$$(p\,r - P_0) = \frac{2 \sin \phi_1}{D_0} \left(p\,r \frac{k^2}{r^2}\right)$$

Values of $\frac{2 \sin \phi_1}{D_0}$.

$2\phi_1$	$\frac{t}{r}$							
	0.00	0.02	0.06	0.10	0.15	0.20	0.25	0.30
*40°	3 021	2 750	1 600	872	461	278	184	130
42.5	2 380	2 200	1 380	806	442	273	181	129
*45	1 884	1 776	1 215	745	424	268	178	128
47.5	1 485	1 435	1 050	672	403	258	175	126
*50	1 235	1 188	909	618	381	248	171	124
52.5	1 010	980	780	558	357	237	166	122
*55	843	821	678	503	334	227	161	119
*60	595	584	508	404	288	205	150	113
65	445	440	386	320	246	182	137	105
*70	320	317	294	256	205	160	125	98
75	246	244	230	203	172	140	112	89
*80	187	186	180	163	141	119	98	81
85	150	149	142	130	117	101	86	72
*90	117	116	113	107	97	86	75	65
95	92	91	89	87	80	72	64	57
*100	76	76	74	72	67	62	56	50
105	64	64	63	62	58	54	48	44
110	54	54	53	52	50	47	42	39
115	44	44	43	42	41	39	36	34
*120	36	36	36	35	34	33	31	29
125	31	31	30	30	29	28	27	25
130	26	26	25	25	24	24	23	22
*140	19	19	19	19	19	18	18	17
150	15	15	15	15	14	14	14	14
160	12	12	12	11	11	11	10	10
170	9	9	9	9	8	8	8	8
*180	7	7	7	7	7	7	6	6

NOTE.—The starred rows were computed; the others were interpolated.

It will be recalled that $(p\,r - P_0)$, as computed by Equation (10) or by use of the coefficients given in Table 6, includes the influence of moments

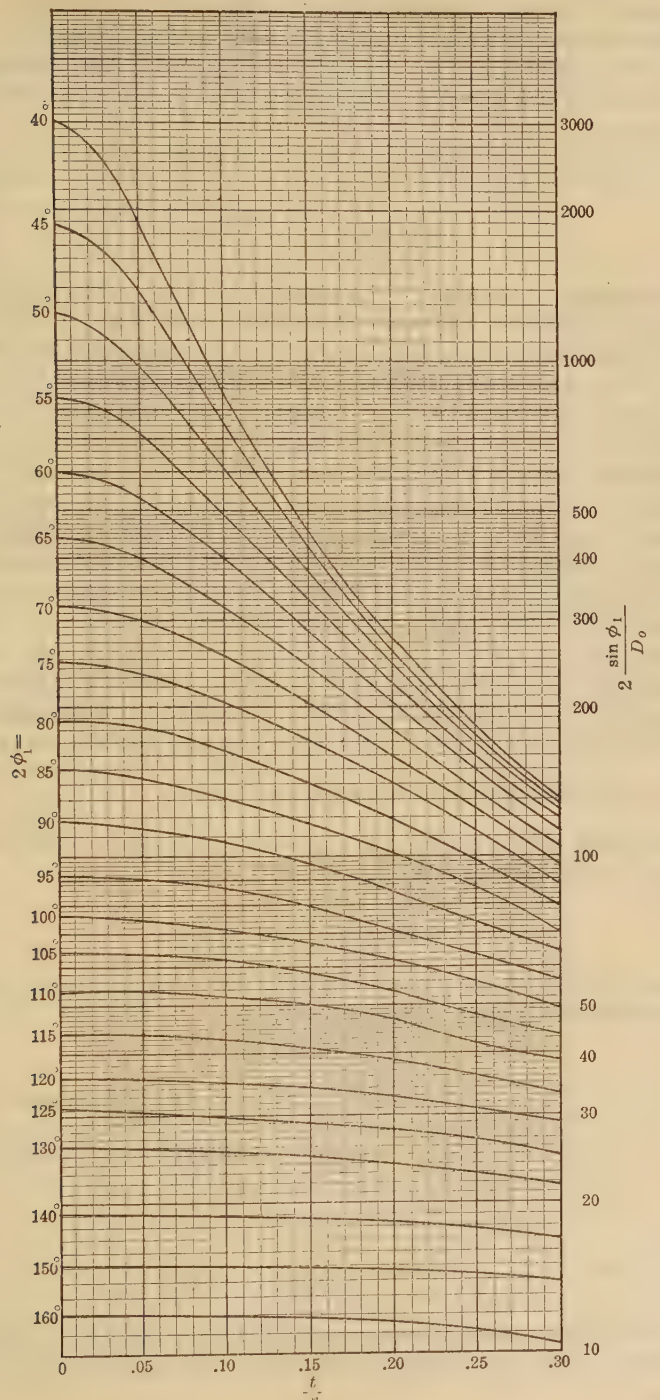
$2\phi_1 =$
40°
45°
50°
55°
60°
65°
70°
75°
80°
85°
90°
95°
100°
105°
110°
115°
120°
125°
130°
140°
150°
160°
3000
2000
1000
500
400
300
200
100
50
40
30
20
10
$2\frac{\sin\phi_1}{D_o}$
0
.05
.10
.15
.20
.25
.30
$\frac{t}{r}$

FIG. 10.

and tangential stress, the influence of shear being disregarded. It was originally assumed that the action of the right half of the arch on the crown section of the left half, was equivalent to a tangential thrust, P_o, and a couple, the moment of which was M_o. The value of this moment, as given by Equation (12), is,

$$M_0 = -(p\,r - P_0)\,E\,C,$$

where,

$$E\,C = r\left(1 - \frac{\sin\,\phi_1}{\phi_t}\right).$$

This moment is, therefore, always negative. The corresponding couple is left-handed, so that the original forces at the crown, Fig. 11 (*a*), can be replaced by the forces shown in Fig. 11 (*b*). On cancelling the two forces, $P_0 - P_0$, at C, there is left the single tangential force ($p\,r$), acting to the left at C, and the single force ($p\,r - P_0$), acting to the right at E, (Fig. 11 (*c*)); and these two forces are exactly equivalent to the original couple (the moment of which is M_0) and the tangential thrust, P_0, both acting at the crown.

It can be shown similarly, for an arch hinged at the ends, that the couple, the moment of which is M_o and the thrust, P_o, both acting at the crown section, can be replaced by a single force ($p\ r - P_o$), acting to the right along the span line, $A\ B$, Fig. 11, and a single force ($p\,r$), acting to the left, at the center of the crown, in a tangential direction.

Messrs. Jakobsen and Mensch have called attention to the desirability of ascertaining the influence of shear, especially for thick arches and small central angles. Consequently, it has been thoroughly investigated, and some numerical results will shortly be given to show the decrease in moment due to shear for various values of $2\,\phi_1$ and ratio of $\frac{t}{r}$.

The formula for the elastic work due to shear by Bach (as quoted by Mr. Jakobsen), for the half arch, is,

$$2.88\,\frac{r}{2}\int_0^{\phi_1}\frac{S^2\,d\,\phi}{E\,t}$$

For an isotropic body (or one having equal elasticity in all directions), the coefficient 3.00 is used in place of 2.88.

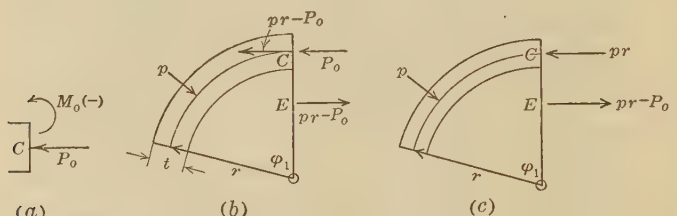

FIG. 11.

On adding the expression just given to the right member of Equation (5), the symbol, L, will then represent the internal elastic work due to moment, thrust, and shear.

The shear, S, as given by Equation (14), is,

$$S = (p\,r - P_0)_s \sin \phi.$$

Therefore,

$$\frac{\delta S}{\delta M_0} = 0; \quad \frac{\delta S}{\delta P_0} = -\sin \phi.$$

Since the shear at the crown is zero, there are only two unknowns, M_0, P_0, to determine. By the method of least work, these can be found by elimination between the two equations:

$$\frac{\delta L}{\delta M_0} = 0, \quad \frac{\delta L}{\delta P_0} = 0.$$

The solution proceeds exactly as before and is very simple. It leads to the formula,

$$(p\,r - P_0)_s = \frac{p\,r}{D_s}\; 2\; \frac{k^2}{r^2}\; \phi_1 \sin \phi_1,$$

where,

$$D_s = D + 2.88\, \frac{k^2}{r^2}\, \phi_1 \left(\phi_1 - \frac{1}{2} \sin 2\, \phi_1\right),$$

D being given by Equation (11).

The subscript, s, indicates that shear is included. The term involving 2.88 as a factor, expresses the influence of shear. When 2.88 is replaced by 3.0, as for an isotropic body, the value of D_s reduces to that given by Mr. Mensch.

When the term involving the factor, 2.88, is omitted, the value of $(p\,r - P_0)$, as given by Equation (10), where shear was neglected, is derived.

Let the moment at the crown when shear is omitted be designated by M_o, and when shear is included, by M_s. Then, since Equations (7) and (12) hold, for the case when shear is included on replacing $(p\,r - P_0)$ by $(p\,r - P_0)_s$,

$$M_s = -(p\,r - P_0)_s \left(1 - \frac{\sin \phi_1}{\phi_1}\right),$$

$$M_0 = -(p\,r - P_0) \left(1 - \frac{\sin \phi_1}{\phi_1}\right),$$

therefore,

$$\frac{M_s}{M_0} = \frac{(p\,r - P_0)_s}{(p\,r - P_0)} = \frac{D}{D_s}.$$

The values of D and D_s were computed for various values of $2\,\phi_1$, t, and r, and their ratio, which is that of M_s to M_o, was inserted in Table 7.

The decrease in the moment at the crown due to shear seems to be negligible for central angles from 90 to 120°, and it only amounts to 10.2% for

$$2\,\phi_1 = 40° \text{ and } \frac{t}{r} = \frac{40}{135} = 0.297.$$

The derivation of the formula for deflection at the crown, when shear is included, involves long and tedious reductions, so that only a few steps will be given. For an arch without hinges, shear being included, by Equation (14),

$$S = (p\,r - P_0)_s \sin \phi$$

When the small radial force, w, is applied at the crown, the shear is,

$$S' = S + w \cos \phi \therefore \frac{\delta S'}{\delta w} = \cos \phi.$$

On adding the work due to shear and proceeding as on page 240, we derive,

$$\frac{E I}{r}\eta = \int_0^{\phi_1} M r \sin \phi \, d\phi + k^2 \int_0^{\phi_1} P \sin \phi \, d\phi + 2.88\, k^2 \int_0^{\phi_1} S \cos \phi \, d\phi.$$

TABLE 7.—ARCH FIXED AT ENDS. RATIO, $\frac{M_s}{M_0}$.

$2\phi_1$	r, in feet.	t, in feet.	$\frac{M_s}{M_0}$	Percentage of decrease due to shear.
40°	135	4	0.979	2.1
40°	135	20	0.909	9.1
40°	135	40	0.898	10.2
90°	300	30	0.952	4.8
120°	300	30	0.971	2.9
120°	135	4	0.997	0.3

After substituting the values of M, P, and S, integrating and reducing, the value of $(p r - P_0)_s$ is eventually introduced, and the equation solved for the deflection, η, giving:

$$\eta = \frac{p r^2}{E t} \frac{(1 - \cos \phi_1)\left\{\left(1 + \frac{k^2}{r^2}\right)(\phi_1 - \sin \phi_1) + 2.88 \frac{k^2}{r^2}(\phi_1 + \sin \phi_1)\right\}}{\left(1 + \frac{k^2}{r^2}\right)\left(\phi_1 + \frac{1}{2}\sin 2\phi_1\right) - \frac{1 - \cos 2\phi_1}{\phi_1} + 2.88 \frac{k^2}{r^2}\left(\phi_1 - \frac{1}{2}\sin 2\phi_1\right)}$$

The influence of shear is represented by the terms containing the factor, 2.88, which, in this case, are found in both numerator and denominator.

As a numerical illustration of the influence of shear, take an arch for which, $2\phi_1 = 60°$, $t = 20$ ft., $r = 200$ ft.; whence, $\frac{t}{r} = 0.1$, $\frac{k^2}{r^2} = \frac{1}{12}\left(\frac{t}{r}\right)^2 = 0.0008333$. When shear is included, the formula gives:

$$\eta = 1.2963 \frac{p r^2}{E t}.$$

On omitting the terms containing 2.88 as a factor, the deflection, when shear is neglected, is found to be,

$$\eta = 1.2772 \frac{p r^2}{E t},$$

or the influence of shear adds 1.5% to the deflection, in this case. From Table 1, for $2\phi_1 = 60°$, $\frac{t}{r} = 0.1$, there is found, $c = 1.277$, which agrees with the coefficient just found, as computed from an equivalent formula.

As to the accuracy of the formulas, since most of the steps have been given, it is easy for any one to verify them. The writer has carefully checked them, and some errors in copying have been noted.* On pages 239 and 243, the second examples given, for $t = 40$ ft., $r = 135$ ft., should have the central

* These errors have since been corrected in the paper.

angle, $2\,\phi_1 = 40°$ and not $2\,\phi_1 = 120$ degrees. Among others, Mr. Mensch has properly called the writer's attention to these two errors.

As Mr. Mensch questions Equation (18), it seems proper to give some of the steps in the reduction, so that any one can easily verify the result. From Equations (16) and (4),

$$\frac{\delta\, M}{\delta\, P_0} = r\,(\cos\,\phi - \cos\,\phi_1);\quad \frac{\delta\, P}{\delta\, P_0} = \cos\,\phi.$$

On substituting these values in Equation (17), and also the values of M and P given by Equations (16) and (4), we derive,

$$r^2\,(P_0 - p\,r)\int_0^{\phi_1}(\cos\,\phi - \cos\,\phi_1)^2\,d\,\phi$$
$$+\,k^2\int_0^{\phi_1}\left\{(P_0 - p\,r)\cos^2\,\phi + p\,r\,\cos\,\phi\right\}\,d\,\phi = 0$$

On integrating and solving for $(p\,r - P_0)$, Equation (18) is readily derived. The denominator was first derived in another form and then reduced to the form given in Equation (18). It is instructive to compute the stresses of the flat arch, Fig. 5, and compare results with those given by Mr. Mensch for a straight beam fixed at the ends. The span, l, is 100 ft., the thickness, t, 40 ft., and the radius of the center line of the arch, as measured, on the drawing, is $r = 140$ ft.; consequently, $2\,\phi_1 = 41°\,50'$, $\frac{t}{r} = 0.286$, and $k^2 = \frac{1}{12}(40^2)$.

Also, from Table 6, $\frac{2\,\sin\,\phi_1}{D_0} = 144$.

Mr. Mensch gives the pressure per square foot on the extrados as 10 000 lb.; therefore, the unit pressure on the center line is,

$$p = 10\,000 \times \frac{160}{140} = 11\,430 \text{ lb. per sq. ft.}$$

Equation (10), or its equivalent, gives,

$$(p\,r - P_0) = \frac{2\,\sin\,\phi_1}{D_0}\,p\,\frac{k^2}{r} = 1\,567\,000;$$

whence, $P_0 = 33\,000$ lb. From Equation (12), $M_0 = -\,4\,840\,000$ ft-lb. The unit stresses at extrados and intrados are found from the formula,

$$s = \frac{P}{t} \pm \frac{6\,M}{t^2},$$

to be, at the crown,

18 970 lb. per sq. ft. compression at extrados.
18 220 lb. per sq. ft. tension at intrados.

At the abutment, the moment, M_1, is found from Equation (13), for $\phi = \phi_1$, to be $M_1 = 9\,613\,000$ ft-lb.; and the tangential thrust, P_1, as computed from Equation (4), is $P_1 = 136\,000$ lb. The resulting unit stresses are;

39 400 lb. per sq. ft. compression at the intrados.
32 600 lb. per sq. ft. tension at the extrados.

For the straight beam, fixed at the ends, the extreme fiber stresses are, $\mp$ 15 000 lb. per sq. ft. at the crown, and $\mp$ 30 000 lb. per sq. ft. at the abutments.

The radial deflection at the crown of the arch is given by the formula:

$$\eta = c \frac{p r^2}{E t},$$

where c, by interpolation from Table 4, is found to be nearly, 0.115.

The deflection for the straight beam, subjected to the uniform loading, $p' = 10\,000$ lb. per sq. ft., can be found by the well known formula:

$$\frac{p' l^4}{32 E t^3} = \frac{p}{E t} \frac{1}{32} \left(\frac{5 r}{7}\right)^2 \left(\frac{l}{t}\right)^2 = 0.10 \frac{p' r^2}{E t};$$

whence,

$$\frac{\text{Deflection of arch}}{\text{Deflection of beam}} = \frac{0.115}{0.100} \frac{p}{p'} = 1.31$$

The computations have been given in full on the hypothesis that the concrete could stand the tension. Since this is not true, the arch would crack, about as illustrated in Fig. 5.* The comments by Mr. Mensch relative to Figs. 6 and 7 are instructive, and the writer is glad to note that he endorses the reinforcing of arch dams and the tying of them in to the rock walls at the abutments. The writer does not think that "stresses can be found from deflection measurements of the arch", where the dam has vertical cracks, unless the deflection and temperature of the dam, at the time of closing of the dam, are recorded. He has made a reservation to that effect in his discussion† of Mr. Noetzli's paper previously mentioned, and, in addition, has referred to the uncertainty relative to the modulus of elasticity.

The subject of inclined dams, commented on by Mr. Mensch, is important, as is known from a study of multiple-arch dams, and it will well repay careful investigation.

Mr. Jakobsen's discussion is not only interesting, but helpful. By taking the span constant and developing the formulas for moments, for an arch fixed at the ends, into series, and, ultimately, taking the limit, as the central angle approaches zero and the radius increases indefinitely, the arch approaches, as a limit, the straight beam with fixed ends, and the formulas for moments at crown and abutments are found to reduce to the corresponding moments for the straight beam at the center and ends. It is a "deadly" test that an exact formula can stand and one that approximate formulas generally fail to meet. It is a test easily made and will doubtless send to the scrap heap a number of formulas confidently proposed. All the formulas for moments and deflections of the writer's paper meet this test; and, to that extent, they may be said to be checked.

* If the uncracked portion of the arch, Fig. 5, is supposed to constitute another arch, the dimensions (as roughly measured from the drawing) will be as follows: Radius, r, of center line = 50 ft., span of center line = 84 ft., thickness of arch, t = 18 ft. The central angle is, consequently, $2\phi_1 = 114° 18'$. On solving as before, it is found that there is no tension anywhere in this new arch, and that the greatest compression is 956 lb. per sq. in. at the intrados at the abutment. The large central angle accounts mainly for the great difference in the results.

† See p. 308.

In the solution, just given, pertaining to the flat arch, Fig. 5, it was found that the stresses at the crown and abutments, as well as the crown deflection, approach the corresponding quantities for the straight beam. Further, the center of pressure at the crown section is $\frac{M_0}{P_0} = -146.7$ ft., from its center, on the extrados side. As the central angle tends toward zero, this center of pressure should recede indefinitely from the crown. To test this, the formula for P_0 was expanded in series, after r was expressed in terms of the span, l, and it was found to approach zero as ϕ_1 decreased indefinitely. Hence, since $M_0 = -\frac{1}{24} p l^2$, when $\phi_1 = 0$, therefore, $\frac{M_0}{P_0} = -\infty$. The same conclusion was reached for the arch hinged at the ends, where the bending moment at the crown, expressed in series, is,

$$M_0 = -p k^2 \frac{1 - \frac{1}{4}\phi_1^2 \dots}{\frac{4}{15}\phi_1^2 \dots + \frac{8k^2}{l^2}\left(1 - \frac{2}{3}\phi_1^2 \dots\right)}$$

This reduces to $\left(-\frac{p l^2}{8}\right)$ for $\phi_1 = 0$, the moment at the center of a straight beam, supported at the ends, when uniformly loaded. It was proved, first, that $P_0 = 0$, when $\phi_1 = 0$, and, again, that $\frac{M_0}{P_0} = -\infty$ for $\phi_1 = 0$. The formulas for deflection afford equally satisfactory checks. Thus, substituting, $r = \frac{l}{2 \sin \phi_1}$ in Equation (15), for an arch fixed at the ends, and expanding,

$$\eta = \frac{p}{E t} \frac{\frac{l^2}{48}\left(1 + \frac{1}{3}\phi_1^2 \dots\right) + k^2 \phi_1^2 \left(\frac{1}{12} + \dots\right)}{\frac{2}{45}\phi_1^2 \dots + \frac{8k^2}{l^2}\left(1 - \frac{2}{3}\phi_1^2 \dots\right)},$$

which, for $\phi_1 = 0$, reduces to, $\frac{1}{32}\frac{p l^4}{E t^3}$, as for a straight beam, when shear is omitted. When shear is included, to compare with the usual formulas, replace the factor, 2.88, by 3.00, in the formula for deflection given previously. To find the deflection due to shear only, use only the terms in the numerator having the factor, 2.88, now replaced by 3.00. After expansion, we derive,

$$\eta = \frac{\frac{3}{2}\frac{p k^2}{E t}\left(2 - \frac{1}{3}\phi_1^2 \dots\right)}{\frac{2}{45}\phi_1^2 \dots + \frac{8k^2}{l^2}\left(1 + \frac{1}{3}\phi_1^2 \dots\right)},$$

which reduces, for $\phi_1 = 0$, to $\frac{3}{8}\frac{p l^2}{E t}$, the exact expression given in textbooks for a straight beam uniformly loaded.

The total deflection, when shear is included, for a straight beam, uniformly loaded and fixed at the ends, is thus,

$$\eta = \frac{p l^4}{32 E t^3} + \frac{3}{8}\frac{p l^2}{E t} = \frac{p l^4}{32 E t^3}\left[1 + 12\left(\frac{t}{l}\right)^2\right].$$

Apparently, the influence of shear, as $\frac{t}{l}$ increases, is more pronounced for the beam than for the arch.

The deflection of an arch hinged at the ends, is given by Equation (19). When this is expressed in terms of l and expanded, there is found,

$$\eta = \frac{p}{E\,t}\,\frac{\frac{1}{2}\left(1-\frac{1}{12}\,\phi_1{}^2\cdots\right)\left\{\frac{l^2}{4}\,\frac{\frac{5}{6}}{1-\frac{1}{3}\,\phi_1{}^2}+\frac{1}{6}\,k^2\,\phi_1{}^2\left(1-\frac{\phi_1{}^2}{20}\cdots\right)\right\}}{\frac{4}{15}\,\phi_1{}^2+\frac{2}{3}\,\frac{t^2}{l^2}\left(1-\frac{2}{3}\,\phi_1{}^2\cdots\right)}$$

which, when $\phi_1 = 0$, reduces to,

$$\frac{5}{32}\,\frac{p\,l^4}{E\,t^3},$$

the known formula for a straight beam.

The value of $(p\,r - P_0)$ given by Equation (10) for fixed ends, and by Equation (18) for hinged ends, does not lead to known formulas as $\phi_1 \doteq 0$; but on dividing by $(p\,r)$, putting in the right member, $r = \frac{l}{(2\,\sin\,\phi_1)}$, and developing; then, as $\phi_1 \doteq 0$, $r \doteq \infty$, and for a straight beam, $P_0 = 0$, either equation gives, at the limit, $1 \doteq 1$.

Finally, Equations (10), (15), (18), and (19) were deduced independently for $\phi_1 = 90$ degrees. The results checked with those given by these formulas for this angle.

The writer's paper was prepared for the practical engineer and every care was used to insure accuracy in both the formulas and the tables. They are believed to be accurate for the following reasons:

(1).—The formulas for temperature thrust, H_0, for either fixed or hinged ends, agree exactly with those deduced by Church by a different procedure.

(2).—The quantity, D_0, appears in the denominators of the formulas for H, $(p\,r - P_0)$, M, and the deflection, η, for the arch with fixed ends. The expression designated by B appears, similarly, in the denominators of the formulas for H, $(p\,r - P_0)$, M, and η, for the arch with hinged ends.

(3).—As the central angle diminishes, the formulas for moments and deflections for an arch with a fixed span—whether with fixed or hinged ends—approach indefinitely the corresponding formulas for a straight beam, and the center of pressure at the crown recedes indefinitely.

(4).—As the central angle diminishes, the two members of the formulas for $(p\,r - P_0)$ approach an identity as a limit and P_0 tends to zero.

(5).—The formulas for $(p\,r - P_0)$ and for deflection, for both fixed and hinged ends, for the special case, $\phi_1 = 90°$, were derived independently. The results agreed with those given by the general formulas for $\phi_1 = 90$ degrees.

(6).—As to the tabular quantities, the regularity of the corresponding curves is the most convincing proof of their accuracy, though a few check

computations were made by use of the formula, written in a different but equivalent form.

(7).—The coefficients of Table 2, when expressed in terms of $\frac{t}{h}$, are found to agree fairly well with those of Mr. Noetzli's Fig. 5, previously mentioned, thus affording a mutual check of results. Also, the coefficient of Mr. Noetzli's formula for deflection gives a rough check to many of the figures of Table 1. The computations of the quantities in Tables 1 and 2, and 4, 5, and 6, was a formidable task, but a systematic procedure lightened the labor and insured greater accuracy.

Mr. Mensch states that he "questions Equation (18) and some of the deflection formulas and many of the figures in Table 1". It is to be hoped that the reasons given in favor of their accuracy, may incline Mr. Mensch to check the various steps in the analysis; when, perhaps, all doubts as to their accuracy may be removed. When possible, it is desirable for the sake of scientific verity, that objections should be specific, so that the author may know what point to meet. In this way, the truth is more quickly attained.

Mr. Jakobsen seems to be under the impression that the expanded formulas may be best for computing moments. The writer has not found that to be the case for any of the formulas. After deriving the formulas for thrusts, moments, and deflections, they were at once expanded, in hope, that, by using the expanded form, numerical computation would be easier; but it was found that too many terms of the series would have to be taken to give sufficient accuracy and the method was abandoned.

It may seem superfluous to some that the writer has introduced into the discussion of curved dams the arch hinged at the ends. B. A. Smith,* M. Am. Soc. C. E., says:

"Unless extraordinary precautions are taken to fix in direction the ends of the arch, A and B, there will be deflections at these points which will correspond very nearly to the deflections due to no bending moment at A and B".

If there is sound rock on the hillsides at the ends of the successive horizontal arches, and these ends are fixed to the hillsides with the same care as is used with a reinforced concrete bridge, then any arch may be regarded as "fixed at the ends". It would be instructive if constructors would state the actual conditions. It is very doubtful if "extraordinary precautions" to fix the ends of the arches are generally taken, and a yielding abutment will invalidate any theory.

Some years ago, Professor Malverd A. Howe, M. Am. Soc. C. E., constructed an experimental reinforced concrete arch† to test the reliability of his summation formulas. The measurements were of such a refined character that H, V, and M, at an abutment, could be accurately ascertained; but, at first, the measurements did not correspond with the computations for an arch fixed at the ends. Although the fixed abutment was of the usual design, it was eventually found to move; but the movement was so small that it was

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 2030.

† See description of arch, apparatus, and measurements, with discussion of results, in *Railway Age Gazette*, March 26th, 1909.

difficult to determine that it did move. However, when the abutment was thoroughly tied to the masonry walls of the house by rods, the larger errors were corrected, theory and observation were reconciled, and the results were regarded as very satisfactory.

For arched dams, the theory for hinged ends may only apply to very thin arches; but, if the thicker arches are only loosely connected with the hillsides, the moments, thrusts, and deflection may be intermediate in value to those for fixed and hinged ends.

It is well to state again what was emphasized on the first page of the writer's paper, that it is the part of the normal load on any horizontal arch, as determined by a correct analysis, that figures in all the computations. Thus, for water pressure, p' (except at one point) is not the unit pressure due to the depth of water at the arch on the extrados, but it is supposed to be the value given by Mr. Smith's analysis, or by Mr. Noetzli's tentative method, when applicable. We have then, p = normal unit pressure on the center line of the arch $= \frac{p'\,r'}{r}$. The value of p at the crest, where the water pressure is nil, is often very large.*

Thus, for the Wooling Dam fixed at the base, with reservoir full, the greatest unit stress in a horizontal arch is at the crest, where $p' = 363$ lb. per sq. ft.; whereas, for the same dam, not fixed, but simply supported, at the base, the greatest unit stress is in an arch about 10 ft. below the crest, where $p' = 755$ lb. per sq. ft. The full water pressure at the base of the dam is 2 062 lb. per sq. ft., and it is seen that nothing like this pressure is exerted on any arch of the dam. Another caution refers to temperature stresses. The horizontal thrust, H, as deduced in this paper for a temperature change, pertains only to an arch perfectly free to move except for the restraints at the abutments. For the actual horizontal arch of a curved dam, the movement of the arch under a temperature change is resisted by the dam acting as a cantilever. Both influences must be considered for an arch regarded as a part of the dam. This is done in the notable paper of Mr. B. A. Smith on "Arched Dams"†, and also in the discussion by the writer of Mr. Noetzli's paper on "Gravity and Arch Action in Curved Dams".‡ The analysis is approximate and not as exact as that pertaining to water pressure.

On the basis of this analysis,§ if p' = normal unit pressure on the extrados of the horizontal arch considered, from the water load and temperature change combined, and using Smith's formula for deflection, the total deflection, D, in feet, due to both causes, is,

$$D = \frac{3}{2}\,\frac{p'\,r'^2}{E\,t} - e\,r'\,t_0,$$

* See the writer's discussions in *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 74, 88; also p. 310.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 2027.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 79 and 90.

§ *Loc. cit.*, pp. 81 to 87.

in which,

e = coefficient of expansion for 1° Fahr. —0.0000055; and

t_0 = change of temperature, in degrees, positive for a rise, negative for a fall of temperature.

By using the writer's more general expression for deflection, where the coefficients, c, may be taken from Table 4 or Table 5, according as the arch is regarded as fixed or hinged at the ends, we have,

$$D = c\,\frac{p\,r^2}{E\,t} - e\,r\,t_0,$$

where $p = \frac{p'\,r'}{r}$, is the normal pressure, in pounds per square foot, on the center line of the arch, r being the radius of that center line, as before.

On solving for $(p\,r)$,

$$(p\,r) = \frac{E\,t}{c\,r}(D + r\,e\,t_0),$$

t_0 being positive for a rise, and negative for a fall, in temperature. For an arch that is not cracked, if D is an observed deflection, the formula will give the value of $(p\,r)$, provided the arch is in compression. It will tend to give confidence in the results if, for observed deflections, computed deflections are substituted, as for the thoroughly worked out case of the Wooling Dam, simply supported at the base.*

The computed values are as follows, for the crest: Water pressure: deflection = 0.362 in.; T = 94 600 lb. per sq. ft.; temperature fall of 20° Fahr.; deflection = 0.270 in.; T = 23 170 lb. per sq. ft. Both deflections are down stream and compression is exerted in both cases.

The combined deflection is thus $D = \frac{0.632}{12}$ ft., and the combined tangential thrust = 117 770 lb.

The computations were based on Smith's formula and value of $c = 1.50$; hence, to compare results properly, write r' for r, or, what amounts to the same thing, in the last formula, let r = 135 ft.; then, for t = 2.2 ft., $t_0 = -20°$, $E = 144 \times 2\,000\,000$ lb. per sq. ft., we derive,

$$(p\,r) = 118\,300 \text{ lb.},$$

which compares very well with T = 117 770 lb., the thrust for the combined water pressure and temperature change.

In the discussion by the writer, previously cited, the formula for deflection, including the cases where T is either tension or compression, is treated fully to prove that the last formula is general if attention is paid to the signs: D is positive when down stream and t_0 is as indicated. The applications to the Barren Jack Dam and the Salmon Creek Dam will show the method to follow when deflection observations are given, provided there are no cracks and the arch is in compression.

* The revised computation for this dam, using 8-place logarithmic tables, is given in the writer's discussion of Mr. Noetzli's paper on "The Relation Between Deflections and Stresses in Arch Dams", p. 310.

In case of vertical cracks, observations should be made of temperature and deflection at the time of the complete closing of the cracks. Then, similar observations are to be made at a subsequent period, and the difference in the temperatures will represent t_0 and the difference in the deflections will give the value of D to insert in the formulas. The uncertainty with regard to what value to assign to E, and the fact that the temperature of a dam is not uniform, particularly for thick dams, will render only an approximate solution in the general case. However, if the deflection observations are taken at the same time as the temperature, a large part of the uncertainty is eliminated, particularly if the air temperature, at the time of the two observations, has not varied for several days at the periods considered. From the value of $(p\,r)$, as computed for any dam, since r is known, the value of p can be formed. If this happens to be greater than, or even nearly as large as, the unit water pressure at the base of the dam, the results should be rejected, for it would indicate, either that there were vertical cracks in the dam, that there was a lack of sufficient precision in the observations, or that the temperature was far from being uniform throughout the dam.

To complete the analysis, reference will now be made to Table 3, with the accompanying formula for the moment, M_0, at the crown of the horizontal arch considered:

$$M_0 = -\,a\left(\frac{E\,t^3}{r\,h}\right)\eta,$$

where the deflection, η, is positive for either water pressure or a fall of temperature, and negative for a rise of temperature.

Continuing the solution for the Wooling Dam, supposed not to be fixed at the base,

Let,

$$\eta_w = \text{deflection for water load} = \frac{0.362}{12}\text{ ft.},$$

$$\eta_t = \text{deflection for } 20^\circ \text{ fall of temperature} = \frac{0.270}{12}\text{ ft.},$$

therefore,

$$\eta_w + \eta_t = D = \frac{0.632}{12}\text{ ft.},$$

and,

$$\eta_w = 0.573\ D,\ \eta_t = 0.427\ D.$$

Suppose the very thin arch to be regarded as hinged at the ends; then, from Table 3, for $2\,\phi_1 = 120^\circ$, $a = 0.083$ for water pressure, and $a = 0.121$ for temperature change, $t_0 = -\,20^\circ$, therefore,

$$M_0 = -\frac{E\,t^3}{r\,h}\left[0.083\,\eta_w + 0.121\,\eta_t\right] = -\frac{E\,t^3}{r\,h}\times 0.1\ D.$$

On substituting, $E = 288\,000\,000$, $t = 2.2$, $r = 135$, $h = 67.5$, $D = 0.0527$, we derive,

$$M_0 = -\,1\,774\text{ ft-lb.}$$

For a dam, for which η_w and η_t have not been computed, and only D is given by observation, the values of η_w and η_t, in terms of D, will have to be

estimated. Some information for such an estimate can be had from the computed values of the Wooling Dam. From Equation (16), for the Wooling Dam, hinged at the ends, since $M = M_0$ when $y = h$,

$$(p\,r - P_0) = \frac{-M_0}{h} = \frac{1774}{67.5} = 30 \text{ lb.}$$

therefore,

$$P_0 = p\,r - 30 = 118\,300 \text{ lb. (say).}$$

The unit stresses at the crown are:

390 lb. per sq. in., compression at the extrados.
358 lb. per sq. in., compression at the intrados.

To effect the solution for the arch fixed at the ends, the proper values of a must be taken from Table 3 and the value of M_0 computed as before. Then, from Equation (12), $(p\,r - P_0)$ can be found and the value of P_0 computed. If desired, the value of M at the abutment can be found from Equation (13) by placing $\phi = \phi_1$.

The deflection lines for the Barren Jack Dam are given in Fig. 1* of Mr. Noetzli's paper on "The Relation Between Deflections and Stresses in Arch Dams," the greatest deflection of 0.58 in. occurring on May 25th, 1909, at the crest, with reservoir full, the air temperature being 47° Fahr. On December 24th, 1908, with an empty reservoir, the temperature was 87° Fahr., and the deflection zero at the crest. We have, therefore, $D = \frac{0.58}{12} = 0.0483$ ft., $t_0 = -40°$; and, using the dimensions given by Mr. Noetzli, $r = 79$ ft., $h = 24$ ft., $t = 2$ ft., whence $2\,\phi_1 = 91°\ 45'$.

Assuming this thin dam to be hinged at the ends, from Table 5,

$$c = 1.58,\ r\,e\,t_0 = -79 \times 0.000\,0055 \times 40 = -0.01738,$$
$$E = 144 \times 2\,500\,000 \text{ lb. per sq. ft.}$$

Therefore,

$$(p\,r) = \frac{E\,t}{c\,r}(D + r\,e\,t_0) = 0.0309\,\frac{E\,t}{c\,r} = 125\,800 \text{ lb.}$$

From Table 3, the coefficients to be used are 0.083 and 0.121, respectively, so that if it is assumed, arbitrarily,

$$\eta_w = 0.4\,D,\ \eta_t = 0.6\,D \therefore (0.083\,\eta_w + 0.121\,\eta_t) = 0.106\,D.$$

Therefore,

$$M_0 = -\frac{E\,t^3}{r\,h}(0.106\,D) = -7\,780 \text{ ft-lb.}$$

$$p\,r - P_0 = \frac{-M_0}{h} = 324 \therefore P_0 = 125\,500 \text{ lb.}$$

The corresponding stresses at the crown of this arch are,

517 lb. per sq. in. compression at the extrados.
354 lb. per sq. in. compression at the intrados.

If the modulus, E, is reduced, the stresses will be reduced nearly as the ratio of the two moduli.

The deflection lines of May 25th, 1909, and of August 10th, 1909, for this dam, are crossed, which throws doubt on the accuracy of the observed deflec-

* See p. 295.

tions. Of course, if there were any cracks not entirely closed, at the time of the observation, when the reservoir was empty, which was probably the case, the results given are misleading and should be rejected.

With regard to the supposed flow or plasticity of concrete, and its varying modulus, it is to be hoped that a curved dam will be built so thoroughly reinforced and tied to the hillsides, that no cracks can form. Then, if precise observations of deflections and temperatures are taken, it should be possible, by the use of the given formulas, to obtain reliable results, provided the value of E is known. To determine that would require a complete solution of the dam, such as has been effected for the Wooling Dam, so that the value of p for any horizontal arch is known; then on solving the equation,

$$D = c\,\frac{p\,r^2}{E\,t} - e\,r\,t_0,$$

for E, its numerical value could be ascertained. It is preferable, of course, to eliminate temperature change by using observations for deflections, for different heights of water, when the dam is at the same temperature.

It has been stated that the solution of Mr. B. A. Smith,* pertains strictly to thin dams. This is evident from the derivation of Equation (1) of the paper cited,

$$\frac{d\,F}{d\,z} - \frac{T}{R} + p = 0$$

When the shear, F, between the successive horizontal arches, is ignored, this reduces to the ordinary cylinder formula, $T = p\,R$, which is only true for the linear arch, or practically true for a thin arch. In a number of examples of thick arches with small central angles, discussed by the writer, as the arch represented by Fig. 5, it has been seen that the cylinder formula is totally inapplicable.

As such thick arches, with small central angles, only occur near the base in a V-shaped valley, it is possible that Mr. Smith's method may lead to practical results. No other rational method has as yet been proposed for the dam not fixed at the base. For the dam fixed at the base, the tentative method will apply, since the coefficients, c, of Tables 4 and 5 take account of the deflections, as $\frac{t}{r}$ and the central angle vary. This is true if the shear between the horizontal arches can be practically ignored, which can be done, as proved by the writer, for thin dams, such as the Wooling Dam.

Formulas for the deflection of the circular arch, for fixed ends, and also for hinged ends, for a rise of t_0 degrees Fahr., have been given on page 247. Mr. A. E. Greene has suggested to the writer, that, in deriving the formulas, "no account was taken of that part of the deflection of the crown which takes place without any work being done on the structure when the temperature changes. * * * If the arch was on rollers, the crown deflection would be $h\,e\,t_0 = r\,e\,t_0\ (1 - \cos\,\phi_1)$ and this should be added to the deflections given." The point is well taken, and it is perhaps well to make the reasoning as clear as possible and then effect the corrections. For

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

the case of the arch with fixed ends, regard the arch temporarily as fixed at the right end, but free to move horizontally at the left end. Thus (1), under the rise of temperature, the arch being free to expand, the rise, h, will be increased by $h\,e\,t_0$, and the free end will move to the left a distance, $2\,r \sin \phi_1\, e\, t_0$. Then, for this expanded arch (2), a horizontal force, H, acting inward and a couple, M_t, are supposed to be applied at the free end, of such magnitudes that the section there is brought back to its original position. In this second stage, work is done, and the solution of the corresponding work equations give the values of H and M_t, or of H and M_0; and the corresponding formula for crown deflection corresponds only to this increase in h due to the forcing back of the free end of the expanded arch to its original position; therefore, the total deflection at the crown must equal this value derived by aid of a work equation added to the deflection for the free arch.

Since deflections away from the center have been assumed to be negative, the deflection, η_1, given on page 247, corresponding to the second stage previously mentioned, is really negative, so that $-\,h\,e\,t_0$ must be added to it to give the total deflection, which is, of course, negative for a rise of temperature.

After the (algebraic) addition and reduction, it is found that the total deflection for the arch with fixed ends, is,

$$\eta = -\,c\,(r\,e\,t_0),$$

where c is the coefficient of $\dfrac{pr^2}{E\,t}$ in Equation (15) and its numerical value for various values of $2\,\varphi_1$ and $\dfrac{t}{r}$ can be obtained from Table 4.*

It can next be shown that the value of a in the formula,

$$M_0 = -\,a\,\frac{E\,t^3}{r\,h}\,\eta,$$

---

*Another solution can be briefly indicated. For the free arch under the rise of temperature, the unit elongation along or parallel to the axis is $e\,t_0$. Suppose (1), this elongation is caused by a tangential tension, P', therefore,

$$P' = t \times E\,e\,t_0 = \frac{E\,I}{k^2}\,e\,t_0,$$

When (2), the expanded arch is brought back to its original position at the abutments by the forces, H and Mt, the total tangential component at $r\,\phi$ is,

$$P = H\cos\phi - P' = \frac{E\,I}{r^2}\,(e\,t_0)\,\frac{2\sin\phi}{D_0}\cos\phi - \frac{E\,I}{k^2}\,e\,t_0.$$

The value of M, page 245, is,

$$M = -\,\frac{E\,I}{r}\,e\,t_0\,\frac{2\sin\phi_1}{D_0}\left(\frac{\sin\phi_1}{\phi_1} - \cos\phi\right).$$

For an arch subjected to the normal unit loads, p, the value of P is found from Equations (4) and (10), and the value of M from Equations (10) and (13). These values become identical with those for temperature change if one places

$$p\,k^2 = -\,\frac{E\,I}{r}\,(e\,t_0);\ \text{or,}\ p\,r = -\,\frac{E\,I}{k^2}\,(e\,t_0) = -\,E\,t\,(e\,t_0).$$

Since the deflection under the loads, $p = c\,\dfrac{r}{E\,t}\,(p\,r)$, by the reasoning of page 240, the deflection under the temperature rise of t_0 degrees is,

$$\eta = c\,\frac{r}{E\,t}\,(p\,r) = -\,c\,(r\,e\,t_0).$$

Similar results follow for the arch with hinged ends.

for a change of temperature, is given for fixed ends by the formula,

$$a = \frac{1}{6} \frac{1 - \frac{\sin \phi_1}{\phi_1}}{\left(\frac{\phi_1}{\sin \phi_1} - 1\right)\left(1 + \frac{k^2}{r^2}\right)},$$

which is likewise the formula corresponding to water pressure.

Similar reasoning for the arch with hinged ends will show that, under a rise of t_0 degrees of temperature, the total crown deflection is,

$$\eta = - c \ (r e t_0),$$

where c is the coefficient of $\frac{p r^2}{E t}$ of Equation (19), the numerical values of c being given in Table 5 for various values of $2 \phi_1$ and $\frac{t}{r}$.

The coefficient, a, in the formula for M_0, for the arch with hinged ends, whether for water pressure or a change of temperature, is,

$$a = \frac{1}{6} \frac{\sin \phi_1 (1 - \cos \phi_1)}{\sin \phi_1 + \phi_1 (1 - 2 \cos \phi_1) + \frac{k^2}{r^2} (\phi_1 - \sin \phi_1)}.$$

In the formula for M_0, a is always positive, and the deflection, η, is positive for either water pressure or a fall of temperature (t_0 negative) and negative for a rise of temperature (t_0 positive).

It now remains to revise the table of coefficients, a (Table 3), in accordance with the foregoing determinations, the results, as given in Table 8, being true for thin arches.

TABLE 8.

$2 \phi_1$	Fixed Ends.		Hinged Ends.	
	Water pressure.	Temperature.	Water pressure.	Temperature.
60°	0.159	0.159	0.096	0.096
90°	0.150	0.150	0.090	0.090
120°	0.138	0.138	0.083	0.083

It is fortunate that the coefficients for water pressure and temperature are the same, for, now, no uncertainty can exist for the case of combined water pressure and temperature change.

This change in the coefficients pertaining to temperature changes will affect some of the results given in connection with observed deflections of the Barren Jack Dam, but the changes are obvious and will affect the final stresses only slightly, therefore, the corrections are not given either for the examples of this paper or in the discussion by the writer of Mr. Noetzli's paper previously mentioned.

Mr. Noetzli properly calls attention to the uncertainties regarding the modulus of elasticity, temperature changes, and other assumptions on which

the theory of arched dams is based. There are other assumptions made in the analysis, which are only approximately true. Thus, in Mr. B. A. Smith's analysis, the radius of the extrados of a horizontal arch is substituted for that of the center line, so that the results are strictly true only for a very thin arch. Also, the writer has assumed p (the part of the unit normal load carried by the arch) as constant for any one horizontal arch, although it possibly varies from crown to abutment. Again, although the writer proved, for the thin Wooling Dam, that the successive horizontal arches could be regarded as free to slide over each other, yet this hypothesis may not be practically true for thick arches. Thus, the computations for arched dams generally lead to approximate results, the case being analogous to that of trusses where secondary stresses are either ignored or allowed for under a sufficient factor of safety. However, in view of all these uncertainties in the assumptions, it is well to use as accurate an analysis as possible, so as not to add other uncertainties by a solution that is only roughly approximate.

Mr. Noetzli refers to the possible marked influence of shear on the deflections for a thick arch. It has been fully discussed by the writer, in what precedes, for the arch with fixed ends. Corresponding formulas were also derived for the arch with hinged ends, but it was not thought worth while to give them.

Mr. Noetzli thinks it questionable whether the thick and flat arches of the lower part of a dam can be properly treated as arches. It all depends on whether the arch can withstand the tension at the intrados or the extrados for the loading assumed. If it can, then the structure acts as an arch for any central angle, however small; and, as previously shown, the formulas reduce to those for a straight beam, similarly loaded, when the central angle is made equal to zero. However, if the computed tension is greater than the material can withstand, then, as illustrated by Mr. Mensch in Fig. 5, another arch will be formed, with a much greater central angle, that can possibly withstand the stresses induced. The computation for the arch, Fig. 5, has been given by the writer in full; and, after the cracking, the results for the new arch are given in a foot-note. The computation for the new arch showed that it did not sustain any tension anywhere and that it could safely carry the load.

Mr. Noetzli states that the results obtained by the use of Figs. 4 and 5* of his paper, "The Relations Between Deflections and Stresses in Arch Dams," check almost exactly with those given by the writer. The figures furnished by the curves of Fig. 4 have not been examined by the writer, but those of Fig. 5 were carefully compared with corresponding figures obtained from Table 2 and the check was found to be satisfactory. Table 2 has been expanded into the more complete form, Table 6, and a corresponding diagram, Fig. 10, added.

In conclusion, the writer returns sincere thanks to those who have so kindly assisted in discussing his paper.

* See pp. 304 and 305.

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THE RELATION BETWEEN DEFLECTIONS AND STRESSES IN ARCH DAMS

By F. A. Noetzli,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Mensch, William Cain, B. F. Jakobsen, Charles P. Dunn, Albert B. Hill, L. Standish Hall, William A. Miller, Chauncy Wernecke, W. H. R. Nimmo, Gardner S. Williams and A. E. Greene, and F. A. Noetzli.

Synopsis

In this paper there is developed a method of calculating the stresses in arch dams, which result from the deformations and corresponding deflections of the arches, due to water pressure, change of temperature, shrinkage, lateral deformation, swelling, etc., provided the deflections of such arch dams have been measured or have been predetermined in some other way.

This method is based on the following fundamental principle: The deflection of an arch of known dimensions and material can be calculated by the elastic theory, if the stresses due to loads or other influences such as change of temperature, shrinkage, swelling, etc., are known. Consequently, if the deflections of the arch have been measured, the stresses which correspond to such deflections can be calculated in the opposite way.

The stresses which in an arch result from the shortening or lengthening of the arch axis, and the corresponding deflections, may be called "arch deflection stresses" to distinguish them from the direct axial stresses resulting from water pressure.

A few examples are given, which show the importance of the arch deflection stresses in some existing dams. These examples emphasize also the great desirability of measuring the deflections of all arch dams in order to indicate ways and means of designing future similar structures on a more economical and also on a safer basis.

Introduction

Although a great many slender arch dams have been built, which have proved beyond any doubt the great superiority of the arch over any other form

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of construction, very few tests have been made to determine the actual stresses and the factor of safety for such structures under different load and temperature conditions. This is all the more astonishing as the cylinder theory on which the design of most arch dams is based, is known to be defective in a great many ways. A comparison between theoretical and actual stresses, therefore, would have been very desirable, but, apparently, most designers were, or had to be, satisfied if the dams withstood successfully the water pressure. It is a fact that up to the present time no arch dam has failed, but this is hardly an excuse for neglecting the collection of data which would be of the greatest value for future constructions.

It is well known that arch dams are statically complicated structures, and any attempt toward a mathematically correct solution of this problem will result either in complicated computations, or it will be futile. With the present knowledge, this problem can be solved by approximations only, and many engineers are still in doubt in regard to the best and safest method of arch dam design. Furthermore, the assumptions on which the design of an arch dam has been based with regard to modulus of elasticity, range of temperature, influence of shrinkage, swelling, lateral expansion, etc., are still more or less uncertain, and can be standardized only by tests and extensive experiments.

These considerations show clearly the great desirability of having deflections and stresses of arch dams measured under different load and temperature conditions. In many other problems, for instance, in hydraulics for the flow of water in pipes, canals, and over weirs, etc., purely theoretical formulas fail, and resort has been had to experiments, whereby coefficients and other data can be determined, to amend the fundamental theory of flow in such a way as to make workable formulas available for all practical cases. In a similar manner, tests and experiments will be necessary to establish a standard method of arch dam design.

Present Theories Used for Arch Dam Design

Ordinary Cylinder Theory.—This is the earliest method of design and by it an arch dam is considered as a theoretical, free cylinder. For the purpose of design, every arch slice between two horizontal planes is considered as free to move relatively to other similar elementary arches.

This method of design of arch dams neglects entirely, first, that the lowest parts of a dam are practically immovable relative to the foundation, and that, consequently, no arching can take place, so that the water pressure at the bottom is resisted entirely by the weight of the dam (gravity or cantilever action); and, second, that the highest arches are stressed and forced to deflect, although theoretically they may not have to support any direct water pressure.

Theoretically, therefore, there are serious objections against designing arch dams by the cylinder formula alone. Nevertheless, it is a fact, that no arch dam has failed yet, and this, with the simplicity of the formula itself, is undoubtedly a strong argument in favor of the cylinder theory. However, arch dams of unprecedented magnitude are being contemplated for the near future. Also, there is a tendency to increase the unit stresses, and this increase may be justified only if the stresses in the arches are calculated more accurately than is possible by the simple cylinder theory.

Monolithic Cylinder Theory.—By this method* of design, an arch dam is considered to be a monolithic cylinder, fixed at the base and along the side-hills. This method, although theoretically as correct as can be expected for such a complicated case and for the assumptions made, is applicable only to some special cases, and its use, therefore, is limited.

Theory of Combined Cantilever and Arch Action.—This method† of designing arch dams is based on the assumption of combined cantilever (gravity) and arch action. By simple graphical constructions the division of the water pressure between vertical cantilever and horizontal arches may be obtained in any vertical section of the dam, that is, at the crown of the arch as well as nearer the side-hills. This method permits also of due consideration being given to shrinkage and temperature deformations, rib-shortening, swelling, lateral deformation (Poisson's ratio), etc., and the stresses resulting from these various influences can easily be determined and taken care of by reinforcement and a suitable distribution of the dam material.

Stresses in Arch Dams Determined from Deflection Measurements

It is a well recognized fact that a definite load acting on a structure produces a definite deflection, the size of which can be determined accurately if the theory of the structure and the basic assumptions underlying such theory are known. For instance, if an arch bridge is loaded with certain loads the deflections of the bridge due to such loads may be calculated by the theory of the elastic arch. On the other hand, if the deflections have been measured at a certain known temperature, the stresses in the material can be calculated with great accuracy, even if the load itself which produces the deflections is not accurately known.

Exactly the same reasoning applies to the relation between deflections and stresses in arch dams. If the load is known, the stresses and deflections may be calculated. If, on the other hand, the deflections have been measured, the stresses which produce such deflections can be determined by calculation. Deflections due to changes of temperature and those resulting from the shrinkage of concrete, etc., may be treated in a manner similar to that used for deflections due to direct loading.

Stresses which occur in the cantilevers, due to deformations and deflections along such beams in a dam, can be calculated by the well known theory of ordinary cantilever beams.

Stresses in the Horizontal Arch Slices Determined from the Arch Deflections

A number of measurements showing the deflections of arch crowns have been made for the Barren Jack Dam in Australia‡ and for the Salmon Creek Dam in Alaska.§ The deflection curves of these dams aroused considerable interest among engineers, but there seemed to be no way to derive any benefit

* "Arched Dams", by B. A. Smith, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

† "Gravity and Arch Action in Curved Dams", by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

‡ *Minutes of Proceedings,* Inst. C. E., Vol. CLXXVIII, p. 1.

§ *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 319.

from these measurements for the future design and construction of similar structures.

As a matter of fact, these deflection curves are of the greatest interest and significance if interpreted mathematically. The method of combined cantilever and arch action provides a way of interpreting the real meaning and the natural consequence of such deflections, and permits of the calculation of the size of the stresses which were necessary to bend and deflect the structure in the manner shown by the deflection curves.

If the length of an arch is increased or decreased by any forces or influences, the arch crown is moved forward or backward accordingly. This produces bending moments if the span of the arch remains unchanged. Consequently, in addition to the axial compression stresses due to the water pressure, bending stresses also occur in an arch dam. For reservoir full, the arches may be considered to be fixed at the abutments; for reservoir empty, the degree of "fixity" is somewhat uncertain, unless the dam is anchored to the rock foundation by reinforcing steel bars.

Approximate Formulas for the Determination of Arch Deflection Stresses

It has been shown by the writer in a previous paper* that a change, ΔL, in the length, L, of an arch slice, 1 ft. thick, produces a horizontal thrust:

$$H = \frac{E \Delta L}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2 \, d L}{I}} = \frac{45 \, E \, I \, \Delta L}{4 \, h^2 \, l} = 0.94 \, \frac{E \, t^3}{h^2 \, l} \, \Delta L \text{ (approximately)} \ldots (1)$$

in which,

H = horizontal arch thrust acting at the distance of about one-third of the rise of the center line measured from the crown;

L = length of center line of arch;

ΔL = change of length, L, due to any cause, such as compression, shrinkage, temperature, etc.;

l = span of arch;

h = rise of arch;

I = moment of inertia of arch $= \frac{1}{12} t^3$;

y = ordinate of any point of the arch axis with regard to coordinates through one abutment of the arch;

t = thickness of arch slice;

E = modulus of elasticity.

For the derivation of Equation (1), deformations resulting from bending moments only were considered. This leads to fairly accurate results for thin arches of high rise. For thick and flat arches, however, the effect of direct and shearing stresses becomes very noticeable. The equation for the thrust, H, is then rather complicated. In analyzing a great number of arches of such dimen-

* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

sions, as they occur in general in arch dams, the writer has found that by considering direct stresses and shear, in addition to bending moments, a fairly accurate value of the total deformation thrust, H, may be obtained by Equation (1a):

$$H = 0.75 \frac{E\, t^3}{h^2\, l} \Delta L \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

The writer believes that for the sake of simplicity the use of Equation (1a) is justified in all average cases for which a maximum degree of accuracy is not of primary importance. Adequate means for determining the deformation thrust, H, more accurately, are given later.

The crown deflection, D, of an arch is given approximately by the equation*:

$$D = \frac{3}{16} \frac{L}{h} \Delta L \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If, in an arch dam, the deflections, D, of the arches have been measured, it is possible to calculate also the corresponding variations, ΔL, of the length of the arch which caused the deflections, D.

Thus, from Equation (2),

$$\Delta L = \frac{16}{3} \frac{h}{L} D \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Introducing Equation (3) in Equation (1),

$$H = 0.75 \frac{E\, t^3}{h^2\, l} \frac{16\, h}{3\, L} D = \frac{4.0\, E\, t^3}{h\, L\, l} D$$

For the arch slices in most arch dams, the length, L, of the arch does not differ greatly from the span, l, and, therefore,

$$H = 4.0 \frac{E}{L^2} \frac{t^3}{h} D$$

and because $\frac{L^2}{h} = 8.3\, R$ (approximately),

$$H = 0.48 \frac{E}{R} \frac{t^3}{h^2} D \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Thus, if the crown deflections, D, of an arch dam have been measured for a number of horizontal arch slices, the horizontal thrust, H, for each of these arch slices of a vertical thickness of 1 ft., is given in pounds by Equation (4) if all dimensions are measured in feet. H is to be taken as plus in the case of a lengthening of the arch, and minus in case of a shortening.

The thrust, H, which acts at a distance of practically one-third of the rise, h, from the arch crown, produces at any point of the arch a bending moment,

$$M = H \times \text{distance of point from the thrust, } H.$$

At the crown,

$$M = H \times \frac{h}{3} = 0.48 \frac{E}{R} \frac{t^3}{h^2} D \times \frac{h}{3} = 0.16 \frac{E}{R} \frac{t^3}{h} D \ldots\ldots\ldots (5)$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 11.

The thrust, H, and the moment, M, combined produce the stresses,

$$f_D = \frac{H}{t} \pm \frac{M}{\text{Sec. Mod.}} = 0.48 \frac{E}{R} \frac{t^3}{h^2} \frac{D}{t} \pm \frac{0.16 \frac{t^3}{h} \frac{E}{R} D}{\frac{1}{6} t^2}$$

$$f_D = 0.48\, D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which D is in feet and f_D is pounds per square foot.

If D is measured in inches and all the other dimensions are given in feet, Equation (6) may also be written in the form,

$$f_D = \frac{D}{25} \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The upper sign in Equations (6) and (7) gives the stresses, f_D, at the intrados, and the lower sign the stresses at the extrados if plus stands for compression and minus for tension. The deflection, D, is thereby to be considered plus for a deflection in an up-stream direction, and minus for a deflection in a down-stream direction.

In a fixed arch, the bending moments at the arch abutments resulting from the thrust, H, are approximately twice as large as at the crown, because the lever arm for H is about twice as long. The stresses are, therefore,

$$f_D = 0.48 \frac{t^3 E}{h^2 t R} D \pm \frac{-2 \times 0.16 \frac{t^3}{h} \frac{E}{R} D}{\frac{1}{6} t^2}$$

$$f_D = 0.48\, D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

for D in feet, or,

$$f_D = \frac{D}{25} \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

for D in inches.

The upper sign in Equations (8) and (9) gives the stresses, f_D, at the extrados and the lower sign the stresses at the intrados if plus stands for compression and minus for tension. The arch deflection, L, is again to be considered plus for a deflection in an up-stream direction and minus for a deflection in a down-stream direction.

As a general rule, it should be remembered that a lengthening of the arch axis (deflection in an up-stream direction) produces a positive thrust, H, and that a shortening of the arch axis (deflection in a down-stream direction) corresponds to a negative thrust, H. This "deflection thrust", H, which acts at a distance of about one-third of the rise of the arch from the crown, therefore, must not be confounded with the axial arch thrust from water pressure.

A deflection of an arch in an up-stream direction (deflection thrust, H, is plus) produces compression at the down-stream side near the crown and at the up-stream side near the abutments. At the same time, tension is produced at the up-stream side near the crown and at the down-stream side near the abutments.

A down-stream deflection (deflection thrust, H, is minus) produces tension at the down-stream side near the crown and at the up-stream side near the abutments, and, further, compression at the up-stream side near the crown and at the down-stream side near the arch abutments.

The arch deflection stresses in any horizontal dam slice for which the crown deflection, D, was measured, may be calculated, therefore, by Equation (6) or Equation (7) for a section at the crown and by Equation (8) or Equation (9) for the abutments. These deflection stresses have to be combined with the direct arch compression stresses from water pressure, in order to furnish the maximum stresses for the arch slice under investigation.

The deflection stress formulas, Equations (4), (6), (7), (8), and (9), should be applied with caution to very thick or flat arches, for which the ratio, $\left(\frac{t}{h}\right)$, is greater than, say, 0.8.

Stresses in the Vertical Cantilevers of Arch Dams

It has been shown previously that, particularly at the time of low dam temperature, practically all the water pressure near the base of arch dams is supported by pure cantilever action. In the upper parts of a dam horizontal arch action generally will prevail, but, nevertheless, the cantilever is forced to be bent and deflected by the respective movements of the arches. Such movements obviously introduce stresses of corresponding size in the cantilever. It is possible to calculate the size of these stresses if the deflections have been measured. The method of doing this is based on the well known theory of stresses and corresponding deflections in cantilever beams, and, therefore, will need no further explanation. A few illustrations, however, will be given in some examples discussed subsequently.

Statical Conditions of Existing Arch Dams

Most of the existing arch dams have been designed and built according to the ordinary cylinder theory. The fact that many of these dams have developed vertical cracks shows clearly that although pure arch action has been assumed in the design, temperature and shrinkage stresses are able at certain times to destroy such action entirely, because no arching can occur through open cracks.

If a reservoir formed by an unreinforced arch dam is empty, it has been shown by experience that in most such dams the opening of the vertical construction joints or other cracks is due to the shrinkage of the concrete during the setting and hardening of the cement and also to a decrease of temperature during the cold months of the year. Such a structure, before any water pressure acts on it, consists under those circumstances of a series of free vertical cantilevers.

Suppose, now, that the water rises slowly in the reservoir. At the beginning, its pressure will be supported entirely by the vertical cantilevers, because the contraction joints and other cracks are open, and it is evident that no arch stresses can be transmitted across open cracks. As soon as the cantilevers have deflected, under the pressure of the rising water, so far in a down-stream direction that all the vertical joints and cracks in the dam are closed, arching may occur and arch pressure is transmitted sidewise to the abutments. In

many existing arch dams, the water had to rise more than one-half the height of the dam before the vertical joints and cracks were closed. For instance, for the Salmon Creek Dam in Alaska, which is a constant-angle arch dam, 168 ft. high, the water had to rise about 110 ft. in the reservoir to be sufficient to keep the contraction joints closed.* In other words, when the water rose for the first time in the reservoir, this dam acted by pure vertical cantilever action until the water had reached a depth of slightly more than 100 ft. Then the contraction joints closed tightly under this pressure, and the arch stresses could aid in supporting the additional pressure when the water rose still higher.

Similar conditions exist for many other slender arch dams, and this is one of the reasons why such structures can be considered with very close approximation as a combination of vertical cantilevers and horizontal arches. Also, for structures which have not developed open vertical cracks visible to the eye, it is well known that contraction of the concrete occurs under the influence of shrinkage and low temperature just as it does in any other concrete structure. Good concrete may carry tension of from 100 to 200 lb. per sq. in., or develop a great number of hair cracks, particularly in reinforced structures. Such conditions again decidedly favor initial vertical cantilevering in the lower parts of such a dam, until either all these hair cracks are closed or the shrinkage tension in the arches is relieved so that actual arch compression can occur. To close the vertical hair cracks or to relieve such shrinkage tension as may exist in a horizontal direction in a dam, however, the vertical cantilever has first to be deflected a corresponding amount. This also shows clearly that in such a dam the water pressure near the base, of necessity, has to be supported almost entirely by vertical cantilever action.

It has been proposed to force grout into the contraction joints "and put initial axial compression into the whole structure, thereby making it act like a solid arch."† Such grouting if it could be extended uniformly all over a crack would be of great advantage indeed. However, serious objections have been voiced against the grouting of contraction joints. Such joints, or other cracks, as is well known, may be $\frac{1}{4}$ or $\frac{1}{8}$ in. wide in the upper parts of a dam, but diminish to nil at or slightly above the foundation. In the lower 50 ft. of a dam of a height of, say, 150 ft., the width of such open joints is, therefore, on an average probably less than $\frac{1}{32}$ or $\frac{1}{64}$ in., and it is more than doubtful whether such fine cracks can be filled evenly by cement grout which does not contain an excessive quantity of water. There exists, therefore, the great danger that large spaces in the joints will remain unfilled and that all the arch thrust due to the pressure of the rising water will be concentrated in those areas which were reached by the grout. This will often result, not only in an uneven distribution of the arch stresses, but also in large eccentricities of the arch thrust, thus producing heavy bending moments which ultimately may become a serious menace to the safety of such a dam.

On the other hand, if only a part of the cross-section of a dam is brought to carry the arch thrust, what help is to be expected from those parts of

* "Improving Arch Action in Arch Dams", by L. R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 320.

† *Loc. cit.*, p. 323.

the structure which lie opposite the unfilled spaces of the contraction joints? Here is the old problem of the case of a chain with some very strong and some rather weak links. If, however, it is recognized that the difficulties of filling contraction joints evenly with grout are practically insurmountable, how should it be possible, as is claimed, to fill these joints under such a pressure as to force the arch crown in an up-stream direction and to put shear on the foundation in the opposite direction to that due to the water load?* The pressure of the liquid grout can in no case be greater than the pressure due to the static head of this liquid, as in the case of any other vessel which is open at the top. The additional pressure shown by the grout pump is simply friction head of the grout flowing in the pipes and through the narrow cracks in the dam. Besides, there remain admittedly free air spaces in the joints, and it is difficult to imagine such air bubbles to be under considerable pressure while they could easily escape toward the open top.

Furthermore, an actual initial stress in the arches and a corresponding movement in the up-stream direction could only occur if the last contraction joint to be filled was widened to make possible the lengthening of the arch corresponding to the supposedly up-stream deflection. For instance, if a dam of the dimensions of the Salmon Creek Dam, referred to previously, should be grouted under such a pressure as to force the crown at the crest for, say, $\frac{1}{2}$ in., in an up-stream direction, the contraction joint under pressure would have to be widened by more than $\frac{1}{2}$ in. Of course, this would release at once the caulking material with which the down-stream side of the contraction joint was closed, and the grout would readily flow out.

It was deemed advisable to discuss somewhat at length the question of grouting the contraction joints of arch dams for the reason that important structures of this kind have been grouted in the past, and similar work is contemplated for the near future. For reasons already given, the grouting of contraction joints and other cracks is considered not only ineffective, as far as putting initial stresses into the arches is concerned, but, in certain cases, it may seriously affect the strength of an arch dam in which, for one reason or another, the joints have not been filled evenly. On the other hand, the construction joints of a straight gravity dam may be grouted with excellent results in order to make such joints water-tight.

A procedure which is undoubtedly much safer than grouting, consists of building or closing the arch during the colder season of the year. If this is not possible, for one reason or another, vertical slots of suitable width should be left during the construction and later filled during the colder season. In both cases, however, proper judgment must be used with regard to the closing temperature, in order that the arches and the cantilever are not over-stressed at the time the arch temperature rises to the seasonal maximum and when the reservoir is empty.

Examples

Practically all existing arch dams, with the exception of the Wooling Dam in Australia, have been designed and built according to the ordinary cylinder

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 326.

theory. As far as the writer knows, the Wooling Dam is the only structure of its kind designed as a monolithic cylinder fixed at the base. The deflections of this dam were measured and a close coincidence between calculated and measured deflections was found, so that the correctness of the monolithic cylinder theory as applied to this dam seems to be established beyond any doubt.

The writer's graphical method of investigating arch dams by considering such structures as acting like a combination of vertical cantilevers and horizontal arches has been proved* to give results which check almost exactly with those obtained by the mathematically correct method given by B. A. Smith, M. Am. Soc. C. E.† Thus, also, the soundness of the theory of combined cantilever and arch action is established, at least with this example. It may be of interest, therefore, to apply those principles to some other dams for which deflections have been measured.

The Barren Jack Dam in Australia.—This is an arch dam of a maximum height of 42 ft. above bed-rock, the radius of the up-stream face being 80 ft. The dam is 5.0 ft. thick at the base and 2.0 ft. thick at the crest. Fig. 1 shows a cross-section and various deflection curves for the arch crown under different load and temperature conditions.

A striking feature of the deflection lines is the comparatively large angle at the foundation between the deflection curves and the line of zero deflection. Undoubtedly, the curves drawn through the various observed points have their origin at one common point at the foundation base. This point of origin, of course, is practically the same for any load and corresponding deflection curves for the simple reason that the bed-rock prevents any appreciable movement of the base of the dam. Elastic deformations are in some ways proportional to the unit stresses in the material, and the unit stresses in the large masses of bed-rock, resulting from the shearing and bending forces exerted by the dam on the rock, are so small at a short distance below the plane of contact between dam and bed-rock that the corresponding elastic deformations of the bed-rock itself are negligible. Any investigation of the bed-rock deformations at the base of any of the existing arch dams will prove this beyond any doubt.

Consequently, it follows that if the bed-rock is not deformed elastically, or in any other manner, for any appreciable amount, the dam may be considered to be rigidly fixed at the base unless cracks should have developed either between the masonry and the rock or in the masonry itself at the time when the dam came under pressure and was deflected more than is elastically possible.

Consider, now, a vertical dam slice at the crown of the arches where the deflections were measured. If such a cantilever was rigidly fixed at the base, either due to its own weight or to special anchorage to the bed-rock, any deflection line would have the original line of zero deflection as a tangent at the origin at the base. The deflection curves of the Barren Jack Dam show a marked angle at the origin between the line of zero deflection and the

* Discussion on "Gravity and Arch Action in Curved Dams", by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 71.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 2027.

deflection curve itself. For instance, the measurements of August 10th, 1909 (Fig. 1), gave the deflection of a point 12 ft. above the base as 0.25 in., and the angle, α, between the deflection line on that date and the line of zero deflection is given by the relation,

$$\tan \alpha = \frac{0.25}{12 \times 12} = 0.00173$$

whence,

$$\alpha = 0^\circ\ 05'\ 58''$$

Such a large angle can only be explained by deformations far beyond elastic ones, or, in other words, by open cracks either between the dam and the foundation or a short distance above the foundation in the masonry itself. That such open cracks may occur due to over-stressing the cantilever, has been indicated previously by other calculations.* The deflection curves of the Barren Jack Dam undoubtedly prove that such a break has occurred in this structure. The maximum arch deflection at the crest was found to be 0.58 in. (May 25th, 1909).

By Equation (7), substituting

$D = -0.58$ in. (down stream, therefore, D, is negative);
$E = 2\,500\,000$ lb. per sq. in. (assumed);
$R = 79$ ft.;
$t = 2$ ft.;
$h = 24$ ft., (estimated);

we obtain the stresses in the arch crown resulting from the deflection, as follows:

$$f_D = \frac{D}{25}\frac{E}{R}\frac{t}{h}\left(\frac{t}{h} \pm 2\right) = \frac{-0.58}{25}\frac{2\,500\,000}{79}\frac{2}{24}\left(\frac{2}{24} \pm 2\right)$$

$$f_D = \begin{cases} -127 \text{ lb. per sq. in. (tension) intrados;} \\ +117 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

On August 10th, 1909, the deflection of the Barren Jack Dam at an elevation of 17 ft. above the base was found to be 0.36 in. (Fig. 1).

By Equation (7), substituting

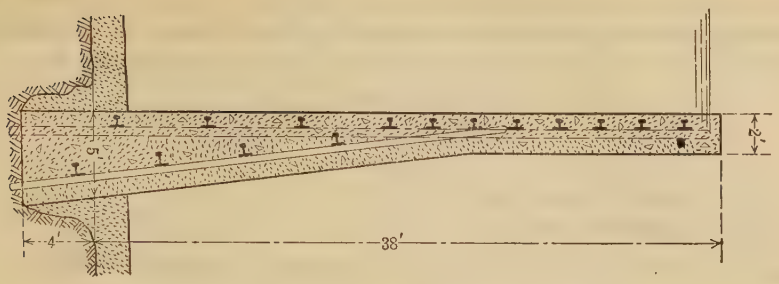
$D = -0.36$ in. (down stream);
$E = 2\,500\,000$ lb. per sq. in.;
$R = 78$ ft.;
$t = 4$ ft.;
$h = 10$ ft. (estimated);

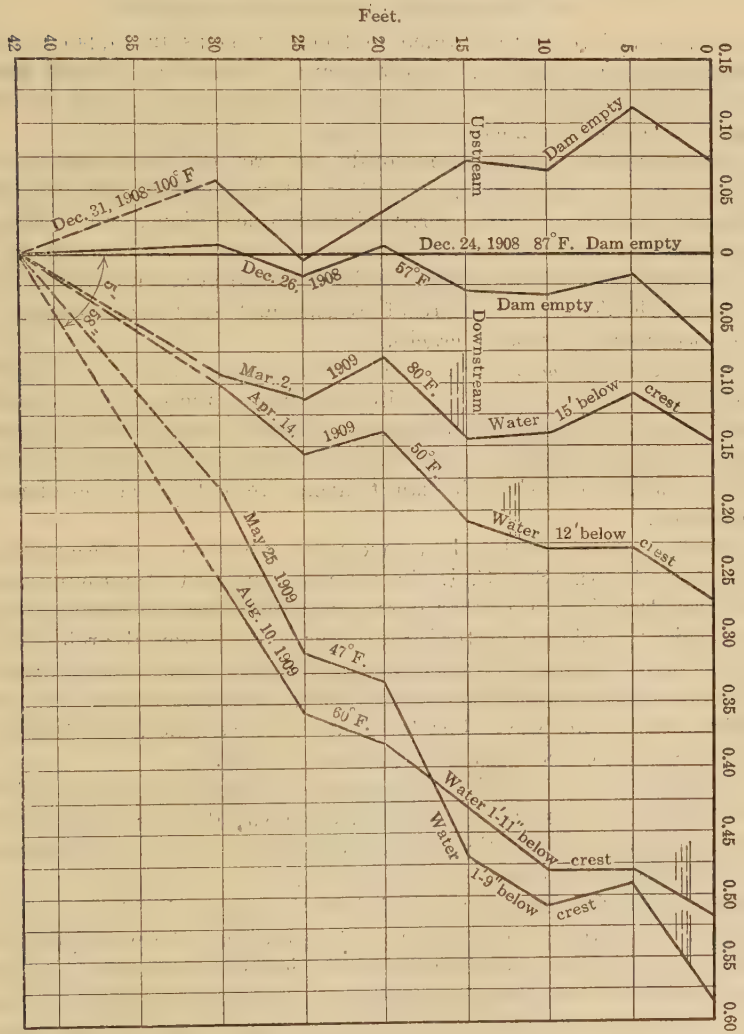
we obtain for the arch deflection stresses at the crown,

$$f_D = \frac{-0.36}{25}\frac{2\,500\,000}{78}\frac{4}{10}\left(\frac{4}{10} \pm 2\right)$$

$$f_D = \begin{cases} -442 \text{ lb. per sq. in. (tension) intrados;} \\ +295 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.





DEFLECTIONS OF BARREN JACK DAM.

FIG. 1.

If the same horizontal arch slice had to support all the water pressure, as is generally assumed when only the ordinary cylinder theory is considered, the compression stresses in the arch due to the water pressure would be:

$$f_w = \frac{P R_u}{t} = \frac{23 \times 62.5 \times 80}{4 \times 144} = 200 \text{ lb. per sq. in.}$$

where

P = water pressure; and,
R_u = up-stream radius

The total stresses in this arch slice at the crown, therefore, would be,

$$f_D = \begin{cases} -242 \text{ lb. per sq. in. (tension) intrados;} \\ +495 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

Due to the cantilever action which undoubtedly helps in supporting the water pressure at this elevation, the direct arch stresses are reduced somewhat. The correct values could be obtained by applying the method of combined cantilever action and horizontal arching.

In a similar manner, as shown previously, the arch deflection stresses and the maximum stresses may be determined at other elevations for any horizontal arch slice and for any characteristic load and deflection in an up-stream or down-stream direction.

It is needless to say that in a scientifically designed arch dam all excessive tension stresses should be taken care of by sufficient steel reinforcement to prevent the occurrence of tension cracks.

The Salmon Creek Dam in Alaska.—The Salmon Creek Dam, near Juneau, Alaska, is the first dam designed and built according to the modern principles of the constant-angle or varying radius types. The deflections of this structure were measured on a number of days when the water in the reservoir stood at different elevations and when the dam temperature had varied considerably from the normal.

The deflection curves of this dam have been published and discussed by L. R. Jorgensen, M. Am. Soc. C. E., in his paper, entitled "Improving Arch Action in Arch Dams".* A further interpretation of these deflections, based on the method of combined vertical cantilever and horizontal arch action, will be given by the writer. It is believed that certain phenomena occurring in the deflections of this dam and not clearly recognized heretofore may be more easily understood in this way. For the sake of convenience, the deflection curves of the Salmon Creek Dam are reproduced in Fig. 2.

The dotted line, *O-A-B,* in Fig. 2, shows the theoretical deflections of the arches for full water pressure and by neglecting cantilever action (simple-cylinder theory), it is evident that the theoretical and the actual deflections do not coincide with each other.

A slight modification in the measured deflection curves has been made in Fig. 2, in that the dotted lower portions of the lines have been brought to a common origin. For reasons explained in more detail in connection with the example of the Barren Jack Dam (page 293), it is believed that the large masses of solid bed-rock into which the base of the dam was keyed, did not deform

* *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

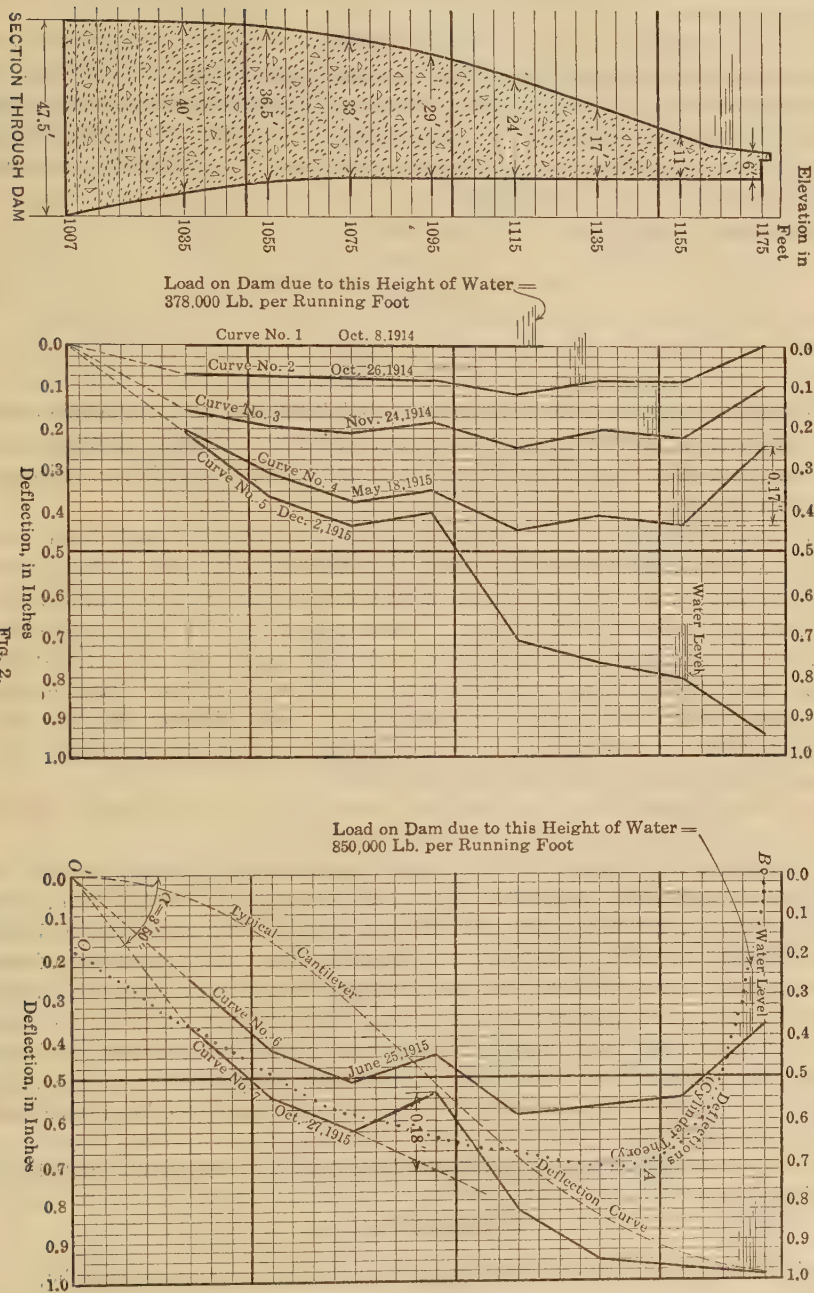


Fig. 2.

DEFLECTIONS OF SALMON CREEK DAM

appreciably under the stresses exerted by the dam, at least not so much as to enable such elastic down-stream deformation to be measured and to be shown on the diagram.

The most striking feature of the deflection lines of this dam is again, as in the case of the Barren Jack Dam, the large angle at the base between the deflection curves and the line of zero deflection. This phenomenon is expressed most decidedly for the deflections measured on October 27th, 1915 (Curve No. 7). At Elevation 1035 (28 ft. above the base), the deflection measured on that day was found to be 0.375 in.

The angle, α, is given by the relation,

$$\tan \alpha = \frac{0.375}{28 \times 12} = 0.00111$$

and,

$$\alpha = 0^\circ\ 03'\ 50''$$

A cantilever fixed at the base and deflected elastically by loads above could not form such a large angle at the base, but would have the line of zero deflections as a tangent, approximately as shown by a typical cantilever deflection curve in Fig. 2.

In view of the fact that the deflections of the Salmon Creek Dam show clearly a large angle between the deflection curves and the line of zero deflection, it is evident that one or a series of cracks have opened in the cantilever on the up-stream side of this structure. These cracks may have occurred either between the dam and the bed-rock or in the masonry itself a short distance above the bed-rock, wherever the least tensile strength existed. Such cracks, of course, will extend only for a comparatively short distance into the dam body, and the water-tightness is insured sufficiently by the increased compression stresses near the down-stream side.

A further proof that this dam has developed horizontal cracks at the up-stream side is shown by the following calculation:

Consider a vertical cantilever slice at the center of the dam, where the deflections have been measured. The measurements of October 27th, 1915 (Curve No. 7), shows the deflection of this cantilever to have been 0.375 in. at a distance of 28 ft. above the base. Neglect, now, for the purpose of this investigation, the portions of the vertical cantilever above Elevation 28. There remains, then, a cantilever of, say, a lateral width of 1 ft. and a depth equal to the thickness of the dam, or, on an average, about 42 ft. in this particular case. The length of this cantilever is 28 ft. and the deflection of the upper end, that is, 28 ft. from the fixed end at the base, has been measured at 0.375 in. (Fig. 3).

In order to deflect such a cantilever for 0.375 in., a uniformly distributed load of about

$$W = 20\,000\,000 \text{ lb.}$$

would be necessary. The bending moment at the fixed end would then be,

$$M = 20\,000\,000 \times \frac{28}{2} = 280\,000\,000 \text{ ft-lb.},$$

producing stresses of,

$$f = \pm \frac{M}{\text{Sec. Mod.}} = \pm \frac{280\,000\,000}{\frac{1}{6} \times 1 \times 42^2} = \pm 950\,000 \text{ lb. per sq. ft.}$$

which, of course, is impossible. Even after consideration of the weight of the cantilever, and all other forces and reactions transmitted from the upper parts of the dam to the short cantilever under investigation, it is clearly seen that such a short and deep cantilever cannot deflect for 0.375 in. without cracking or being partly lifted from the "fixed" end. It is also impossible that the well-seasoned concrete at the base should have "flowed" so much as to permit such a large deflection before the limit of tensile strength of the concrete was reached at the up-stream side.

These mathematical interpretations of the deflections of the Salmon Creek Dam seem to be conclusive proof that this structure broke away partly from the base and developed horizontal cracks at the up-stream side at a time when the water in the reservoir had risen toward the crest and the dam temperature was low.

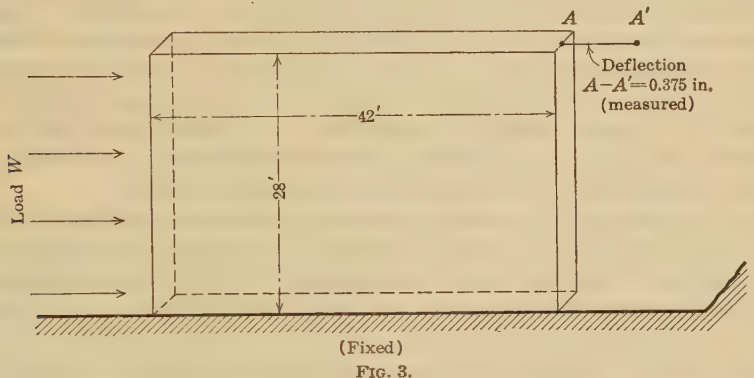


FIG. 3.

Another matter of great interest is the "knee" in the deflection lines at Elevation 1095. This irregularity in the deflections was explained by the fact that the concrete near Elevation 1095 was poured during a period of low temperature (end of construction season). Curve No. 7, Fig. 2, shows the maximum bend in the knee between Elevations 1075 and 1115 to be about 0.18 in.

Any calculations for determining the deflection of a concrete beam, 40 ft. long and 28 ft. deep, will show that a deflection as large as 0.18 in. at the center is impossible for an unreinforced beam of these dimensions without producing cracks. For instance, in order to bend a concrete beam, 40 ft. long, 28 ft. deep, and 1 ft. wide, for 0.18 in. at the center, theoretically, a load of about 9 500 000 lb., uniformly distributed, would be required, producing a maximum tension of more than 2 500 lb. per sq. in. Such is, of course, impossible.

Similar reasoning leads to the conclusion that there may be another horizontal crack in the dam near Elevation 1155. The measurements made on May 18th, 1915 (Curve No. 4, Fig. 2), show that below Elevation 1155

the deflection line was nearly vertical for about 40 ft. and that, at the crest, the dam deflected about 0.17 in. relatively to Elevation 1155. A short calculation for the cantilever portion above Elevation 1155 shows that, even under conservative assumptions, this cantilever, 20 ft. long, with an average thickness of about 8 ft., would have to sustain tensile stresses of more than 900 lb. per sq. in., in order to allow a deflection of 0.17 in. at the free end. This seems to indicate that probably another horizontal break has occurred at or near Elevation 1155, as stated.

The deflection curves of the Salmon Creek Dam, therefore, lead to the conclusion that in this structure the vertical cantilever is broken at three different elevations. The breaks occurred most probably in horizontal construction joints where new concrete was poured on top of the old, or where the accumulation of laitence had reduced the tensile strength. Although the deflection curves clearly indicate the presence of cracks, nothing can be predicted, of course, with regard to their number, width, and extension, without deflection measurements being taken at other points of the dam. It may be argued that the concrete of the dam "flowed" in excess of the elastic deformations and thus prevented the occurrence of open cracks. However, while concrete may "flow" somewhat under high compression stresses, it hardly did so to a marked degree before the limit of the tensile strength, say, 150 to 200 lb. per sq. in., of the concrete of the structure under consideration, was reached. It is now a matter of speculation as to what degree such cracks may affect the safety of the structure and whether there is any danger of gradual disintegration under the occurring reversible stresses.

When the water in the reservoir stands, for instance, at Elevation 1095, where a horizontal crack apparently exists, the cantilever and, therefore, the whole dam has the tendency to be deflected somewhat in a down-stream direction. At the same time, the arch ring from Elevation 1095 upward may be forced in an up-stream direction, say, due to a rise in temperature. Thus, shearing stresses of considerable magnitude occur in the plane of the crack, and this fact may lead to a movement and to progressive deterioration, if the climatic conditions are such as to favor deflections in opposite directions.

It may now be of interest to calculate also the "arch deflection stresses" in the horizontal dam slices.

The maximum deflection of the highest arch, as measured on October 27th, 1915, was 0.98 in. (Curve No. 7, Fig. 2). As a matter of fact, it must have been somewhat larger, because the assumed line of zero deflection (Curve No. 1) includes already the deflection necessary to close the open cracks in the dam.*

By Equation (7), substituting

$D = -0.98$ in. (down stream);
$E = 2\,500\,000$ lb. per sq. in.;
$R = 325$ ft.;†
$t = 6$ ft.;†
$h = 150$ ft.;†

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 320.

† These dimensions are taken from Plate XIII, *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 709.

we obtain the stresses at the arch crown resulting from the deflection, as follows:

$$f_D = \frac{-0.98}{25} \times \frac{2\,500\,000 \times 144}{335} \times \frac{6}{150}\left(\frac{6}{150} \pm 2\right)$$

$$f_D = \begin{cases} -24 \text{ lb. per sq. in. (tension) intrados;} \\ +23 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

On the same day, the deflection of the dam was 0.82 in. at an elevation of 108 ft. above the base (Curve No. 7, Fig. 2). At this elevation,

$$R = 282 \text{ ft.};$$
$$t = 24 \text{ ft.};$$
$$h = 76 \text{ ft.}$$

Then, by Equation (7), the arch deflection stresses at the crown of this arch slice are,

$$f_D = \frac{-0.82}{25} \times \frac{2\,500\,000 \times 144}{282} \times \frac{24}{76}\left(\frac{24}{76} \pm 2\right)$$

$$f_D = \begin{cases} -213 \text{ lb. per sq. in. (intrados) tension;} \\ +154 \text{ lb. per sq. in. (extrados) compression.} \end{cases}$$

The arch deflection stresses at the abutments are found by Equation (9), as follows:

$$f_D = \frac{-0.82}{25} \times \frac{2\,500\,000 \times 144}{282} \times \frac{24}{76}\left(\frac{24}{76} \pm 4\right)$$

$$f_D = \begin{cases} -396 \text{ lb. per sq. in. (extrados) tension;} \\ +338 \text{ lb. per sq. in. (intrados) compression.} \end{cases}$$

These arch deflection stresses which, as must be remembered, are due entirely to the bending of the arch while it was being shortened as the result of axial compression, drop of temperature, shrinkage, etc., have to be combined with the stresses due to the pressure of the water. If all this pressure was carried by horizontal arching (ordinary cylinder theory), the stresses from water pressure in the arch investigated would be:

$$f_w = \frac{58 \times 62.5 \times 294}{24} = +307 \text{ lb. per sq. in. (compression).}$$

This stress combined with the arch deflection stresses gives the maximum unit stresses in this arch slice, as follows:

At the crown..... $\begin{cases} \text{Intrados } f_{max.} = +94 \text{ lb. per sq. in. (compression);} \\ \text{Extrados } f_{max.} = +461 \text{ lb. per sq. in. (compression).} \end{cases}$

At the abutments. $\begin{cases} \text{Extrados } f_{max.} = -89 \text{ lb. per sq. in. (tension);} \\ \text{Intrados } f_{max.} = +645 \text{ lb. per sq. in. (compression).} \end{cases}$

In this manner, the arch deflection stresses and the corresponding maximum stresses in the arching parts of this dam may be determined at any other elevation and for any load and temperature condition for which deflection measurements are available, or may be made any time in the future.

The calculations point to a maximum compression at the intrados near the abutments of slightly more than 600 lb. per sq. in. It is possible that under such high unit stresses the green concrete "flowed" somewhat so that a re-adjustment took place, which reduced the theoretical unit stresses both

on the tension and on the compression side. Such a "flow" could be best detected by continuous deflection measurements at various critical places on the dam. Furthermore, it has to be considered that whenever, in an unreinforced arch dam, cracks have opened on the tension side, the theoretical maximum compression stresses are also reduced, because the resultant of the compression stresses in such a case has to equal the compression resulting from the external loads.

More Accurate Formulas for the Determination of the Arch Deflection Stresses

As previously mentioned, Equations (6), (7), (8), and (9), are only approximations, their main advantage being their simplicity. Greater accuracy is obtained by taking into consideration, in a theoretically correct manner, also the deformations due to the axial stresses and the shearing stresses, in addition to those resulting from bending moments.

The elastic theory of arches, as given in most modern treatises on arch design, shows the horizontal arch thrust, H_f, which is due to rib-shortening resulting from loads applied to the arch, to be:

$$H_f = - \frac{H_a \int \frac{d\,L}{A}}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{A} + 3 \int \sin^2 \alpha \frac{d\,L}{A}} \quad \ldots\ldots (10)^*$$

whereby, in the case of a horizontal slice of an arch dam, 1 ft. thick:

H_f = arch thrust resulting from rib-shortening;
H_a = arch thrust due to the water pressure;
$d\,L$ = length of an arch element;
A = cross-section of arch (= thickness, t, of arch slice);
y = ordinate of arch elements, $d\,L$, with regard to a horizontal X-axis drawn through the center of gravity of the whole arch;
α = angle of inclination between arch element and horizontal X-axis; and
I = moment of inertia of cross-section of arch $\left(I = \frac{1}{12} t^3\right)$.

According to Hooke's law, H_a has the value:

$$H_a = \frac{\Delta\,L}{L} E\,t$$

where E equals the modulus of elasticity and $\Delta\,L$ equals the total shortening of the arch axis. Further, there is,

$$\int \frac{dL}{A} = \frac{L}{t}$$

The numerator of Equation (10) is, therefore, $E\,\Delta\,L$.

The first term in the denominator of Equation (10), for H_f, that is, $\int y^2 \frac{d\,L}{I}$, represents the influence from bending moments; the second term,

* Equation (10) should not be used for arches with central angles of more than, say, 120 degrees.

$\int \cos^2 \alpha \frac{d\,L}{A}$, gives the influence of the direct stresses; and the third term, $3 \int \sin^2 \alpha \frac{d\,L}{A}$, represents the influence from shearing stresses.

The main term in the denominator of Equation (10) is $\int y^2 \frac{d\,L}{I}$, and, as shown in the writer's paper, "Gravity and Arch Action in Curved Dams",* this equals very nearly $\frac{4\,h^2\,l}{45\,I}$. Consequently, H_f has approximately the value,

$$H_f = 0.94 \frac{E\,t^3}{h^2\,l} \Delta\, L$$

which is the same as that obtained in Equation (1).

In order to obtain a more accurate value of H_f, we will compare its value (Equation (10)) with the approximate value given by Equation (19 *a*) of the writer's paper, "Gravity and Arch Action in Curved Dams"*. Equation (10) may be written:

$$H_f = - \frac{H_a \int \frac{d\,L}{t}}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{t} + 3 \int \sin^2 \alpha \frac{d\,L}{t}} \quad \ldots\ldots (11)$$

Equation (19*a*) of the writer's paper,* previously mentioned, reads:

$$H_f = - 0.94\, f'_c \frac{t^3}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which f'_c equals the axial stress due to water pressure. The thrust, H_a, in Equation (11) equals $f'_c\, t$, and the whole numerator, therefore, has the value, $f'_c\, L$.

In order to make H_f in Equation (12) equal to the more correct value of H_f in Equation (11), one simply has to introduce in Equation (12), instead of the approximate constant value, 0.94, such a factor, k_f, that Equation (12) furnishes the same values for H_f as Equation (11).

The factor, k_f, may be calculated from the relation:

$$H_f = - \frac{f'_c\, L}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{t} + 3 \int \sin^2 \alpha \frac{d\,L}{t}} = - k_f\, f'_c \frac{t^3}{h^2}$$

from which,

$$k_f = \frac{h^2\, L}{t^3 \left(\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{t} + 3 \int \sin^2 \alpha \frac{d\,L}{t} \right)} \quad \ldots\ldots (13)$$

The values, k_f, have been calculated for arches with central angles of from 60 to 180° and for various proportions of $\left(\frac{t}{h}\right)$, which are plotted in Fig. 4.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

The arch thrust, H_f, which is due to rib-shortening, may be calculated, therefore, for any arch slice of an arch dam from the following equation:

$$H_f = -k_f f'_c \frac{t^3}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

whereby k_f may be taken from Fig. 4 for the different central angles of the arches and the various proportions of $\left(\frac{t}{h}\right)$.

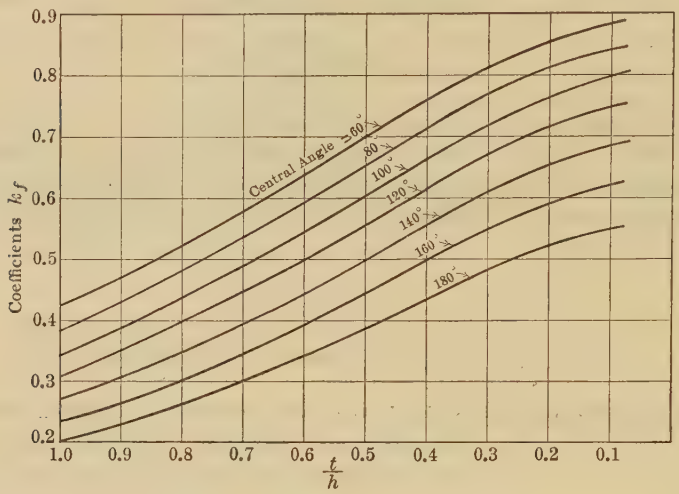


FIG. 4.

The arch thrust, H_f, which acts, as should be remembered, at a distance of approximately one-third of the rise of the arch from the crown, produces bending moments and stresses in the arch. For instance, at the crown, the stresses due to H_f are:

$$f_c = \frac{H_f}{t} \pm \frac{H_f \frac{h}{3}}{\text{Sec. Mod.}}$$

$$f_c = -k_f f'_c \frac{t}{h}\left(\frac{t}{h} \pm 2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

in which, f_c is pounds per square foot; and at the abutments:

$$f_c = \frac{H_f}{t} \pm \frac{H_f \frac{2}{3} h}{\text{Sec. Mod.}}$$

$$f_c = -k_f f'_c \frac{t}{h}\left(\frac{t}{h} \pm 4\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

The deformations resulting from temperature and shrinkage have to be treated separately from those due to rib-shortening.

For temperature deformations, the theory of the elastic arch gives the horizontal thrust:

$$H_t = \frac{c\,T\,l}{\int y^2 \frac{d\,L}{I} + \int \frac{d\,L}{A}} \qquad (17)$$

in which,

c = coefficient of expansion;
T = change of arch temperature;
l = span of arch.

In a manner similar to Equation (14) for H_f, Equation (17) for H_t, may be transformed into:

$$H_t = k_t\,E\,c\,T\,\frac{t^3}{h^2} \qquad (18)$$

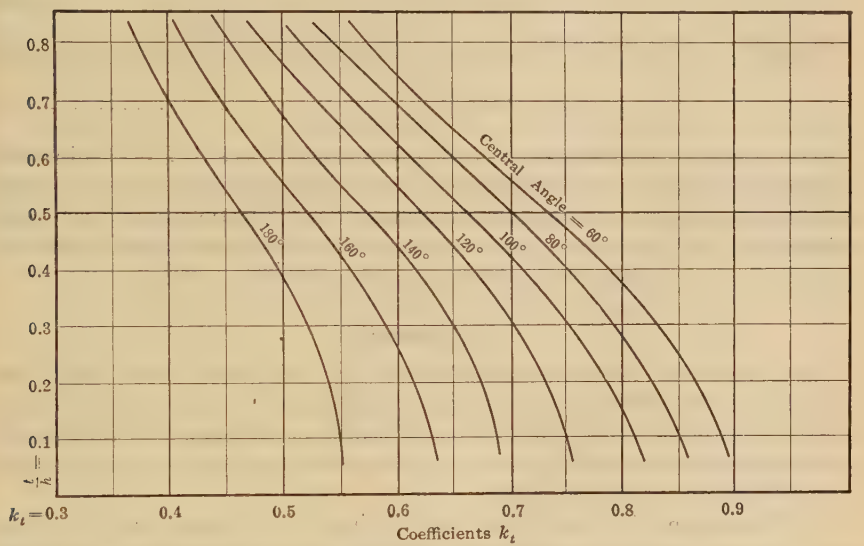


FIG. 5.

The numerical values for k_t are calculated and plotted in Fig. 5 for different central angles between 60 and 180° and for various values of $\frac{t}{h}$.

The stresses in the arch due to H_t are, at the crown,

$$f_c = k_t\,E\,c\,T\,\frac{t}{h}\left(\frac{t}{h} \pm 2\right) \qquad (19)$$

in which, f_c is pounds per square foot; and at the abutments,

$$f_c = k_t\,E\,c\,T\,\frac{t}{h}\left(\frac{t}{h} \pm 4\right) \qquad (20)$$

If the deflections, D, of an arch dam have been measured, and it is desired to know the resulting arch deflection stresses, the procedure is as

follows: Insert in Equation (14) for f'_c (axial stress due to water pressure), its value (Hooke's law):

$$f'_c = \frac{\Delta L}{L} E$$

Then,

$$H_f = k_f \frac{E t^3}{h^2 L} \Delta L$$

Introduce for ΔL its value according to Equation (3),

$$\Delta L = \frac{16 h}{3 L} D$$

The deformation stresses in the arch may then be obtained in a manner similar to that previously shown. At the crown,

$$f_D = 0.64 k_f D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \ldots\ldots\ldots\ldots\ldots (21)$$

in which, D is in feet, and f_D is pounds per square foot; and at the abutments,

$$f_D = 0.64 k_f D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \ldots\ldots\ldots\ldots\ldots (22)$$

The upper sign in Equation (21) gives the stresses, f_D, at the intrados and the lower sign at the extrados, and opposite for Equation (22). Compression is shown by plus and tension by minus. The deflection, D, is to be considered plus for a deflection in an up-stream direction (arch thrust positive), and minus for a down-stream deflection (arch thrust negative).

Conclusions

The results of the investigations given in this paper show clearly the necessity of considering in the design of arch dams not only the direct compression stresses resulting from the water pressure (ordinary cylinder theory), but also those stresses which are due to deformations and corresponding deflections of the arches and cantilevers.

The deflection of an arch dam may be due to any reason, such as direct water pressure, temperature, shrinkage, swelling, lateral deformation (Poisson's ratio), etc., or a combination of any or all of them. The fact that such deflections have been measured enables the determination of the resulting arch deflection stresses, by the aid of the formulas given, with a fair degree of accuracy.

It has been claimed recently that whenever it takes a comparatively long time, say, 14 days or more, to develop the full load in a concrete dam, the stresses, under such circumstances, would be greatly reduced due to the so-called "time factor". It is a fact borne out by tests that "green" concrete, and, to a certain degree, also, older concrete, deforms under high compression stresses somewhat more than the assumed laws of true elasticity would permit. Such test results, however, which were made under conditions absolutely strange to what ordinarily is the rule for arch dams, can hardly be applied directly to dam construction, and, as far as the writer knows, they also are not considered in any other practical design of concrete structures. As a matter

of fact, experience has shown that in dam construction, in an unmistakable manner, the "time factor" is of negligible influence as long as the unit stresses are low, inasmuch as it is not even able to prevent the very slowly working shrinkage deformations from cracking the concrete. It is the writer's opinion that for these reasons the "time factor" should not be considered in the design of arch dams, as long as the stresses are kept below a reasonable limit.

The foregoing deflection-stress formulas may be of value also for the design of new arch dams. It is possible in many cases to predetermine the probable deflections of a dam designed as a "cantilever arch dam" by the method of combined cantilever and arch action. Consequently, the "arch deflection stresses" in such a structure may also be predetermined with a fair degree of accuracy. On the other hand, those formulas will enable an approximate determination to be made of the stresses in the arches of existing dams if the deflections are measured. This will then permit engineers to judge the factor of safety of such dams and, also, to draw conclusions with regard to the stresses that may be expected in future similar structures.

This fact emphasizes most forcefully the great desirability of having deflection measurements made for all arch dams, and of interpreting such deflections mathematically, for instance, in the manner pointed out by the writer. Such investigations and a thorough discussion of the results will then lead to definite and closely defined safe assumptions for the future construction of arch dams, and the result will be not only a great economy in the cost of such structures, but also a uniform and definite factor of safety.

The writer is indebted to P. Bauman, Jun. Am. Soc. C. E., for valuable assistance in calculating the coefficients used for plotting the curves of Figs. 4 and 5.

DISCUSSION

L. J. Mensch,* M. Am. Soc. C. E. (by letter).—During a recent visit to the Pacific Coast, the writer was greatly impressed by the great importance of dams to the prosperity of the western part of the United States. For irrigation, power development, and flood control, dams of enormous heights are proposed, among which may be mentioned the Pacoima Creek Dam, in the San Fernando Valley, north of Los Angeles, 375 ft. high, the St. Gabriel Dam, 15 miles west of Los Angeles, 420 ft. high, and the gigantic Boulder Canyon Dam of the Colorado River, with a height above bed-rock of 730 ft., about 200 ft. long at the base, and 1 000 ft. long at the crest. The sparsely settled West can hardly afford gravity dams of these great dimensions.

Water and power have always been absolute necessities for the development of the West and capital has been scarce, therefore, new types of dams came into vogue, which would have been out of the question in more settled countries. Such dams are the rock-fill dam, the hydraulic-fill dam, the multiple-arch dam, and the single-arch dam (notably represented by the Bear Valley Dam built in 1884 and the Upper Otay Dam built in 1900, both unsurpassed for economy by any other dam, or for safety by few gravity dams). When it is considered that such a simple problem as the design of a gravity dam required more than 100 years of experience and many important failures before it was partly solved so that engineers agreed fairly well as to the main features of the design, the fact that the design of new types of dams is not free from many imperfections need not cause much surprise. These dams are usually designed by hydraulic engineers who have their own particular problems to develop and who, as a rule, have not the time, inclination, nor experience and perception, not to speak of the important knowledge of the literature on the subject, to delve into the intricacies of strange and difficult structural problems.

Engineers trained in structural design look with wonderment, and also with suspicion, at the apparently crude principles by which most of these dams have been designed, and Mr. Noetzli's paper may be considered as a plea against dam design which is not based on strictly structural lines, and more especially against the practice of designing single-arch dams on the principle of a thin cylinder. Scientific principles of the co-related problem of the design of cylindrical tank walls where they join the bottom were published many years ago. Grashof treats of it in Paragraphs 203 to 210 of his book.† A graphical solution is given by Dr. von Emperger‡ for a tank 130 ft. in diameter and 28 ft. high. A very good and simple approximate solution is given on page 257 of *Beton und Eisen* for 1907. Professor Reissner has given a more rigorous treatment of the subject,§ which is nearly identical with the solution presented to the Society twelve years later by B. A. Smith, M. Am. Soc. C. E.‖ Since then, at least a dozen articles on the same subject have appeared in *Beton und Eisen*. An approximate solution was published in *Engineering and*

* Gen. Contr., Chicago, Ill.

† "Theorie der Elasticität und Festigkeit", Berlin, 1878.

‡ "Handbuch für Eisenbetonbau", Vol. 3 (1907).

§ *Beton und Eisen*, 1908, p. 226.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

Contracting in 1912 for a tank built by the writer in Penetanguishine, Ont., Canada.

That large tanks, built of masonry or even of steel, which are not designed for the restraint of the walls at the bottom, are not satisfactory may be shown by the collapse of a large molasses tank in Boston, Mass., a few years ago, and the trouble caused by cracks in concrete tanks where the walls join the bottom. A serious case came to the writer's attention some time ago. A certain Middle Western town contracted for two reinforced concrete tanks 150 ft. in diameter and 24 ft. high. The wall was 24 in. thick at the bottom and was keyed into a recess of the foundation. After the first tank was finished, water was admitted, but the wall sheared off the key and, besides, cracked the floor of the tank near the wall. On the writer's advice a number of circumferential bars near the bottom were omitted in the construction of the second tank, and cut up into shorter bars which served as reinforcement of the joint between the wall and the bottom to take up the cantilever moment at the base. The second tank, although built by the same contractor, was tight from the first filling.

The writer attempted to use the same theory for the design of single-arch dams, and was surprised to find that according to this theory nearly all existing arch dams ought to show serious defects. Although a few such dams in Australia have shown some vertical and horizontal cracks, they evidently are not serious, and few cracks except those which may be ascribed to temperature action have been reported on dams built in the United States. No considerable leakage at the bottom, like that so often observed in reinforced concrete tanks where the walls are not properly tied to the bottom, have been reported for any arch dam; on the contrary, many arch dams are absolutely dry on the down-stream side.

The author's praiseworthy efforts to solve the problem failed, in the writer's opinion, because he tried to solve only that part which has been understood fairly well by structural engineers and which, as has been stated, does not explain the well known safety of arch dams. He tried to explain the arch action by the theory of the thin parabolic arch of small rise, although every textbook containing this theory warns the student not to use it in case of thick arches or arches of considerable rise. Serious discrepancies having been called to his attention in the discussion of his first paper,* he tried to improve his theory by using the formulas of a thick parabolic arch with small rise, which is not sufficiently accurate for the lower arches of a dam. In his excellent paper on thick circular arches† Professor Cain has shown that Mr. Noetzli's formula for deflection is not reliable for flat arches, and the writer can affirm that the curve in Fig. 4 for the 180° arches gives values at least 50% too great and the other curves are in error in proportion. In the upper part of a dam, the secondary stresses are comparatively small and, for that part, the author's formulas give workable results.

How untenable the author's approximate theory is, will be illustrated by the example he chose, the Salmon Creek Dam, designed by L. R. Jorgensen,

* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

† "The Circular Arch Under Normal Loads", p. 233.

M. Am. Soc. C. E. The author has shown that the cantilever broke when the water rose to the top of the dam; he also has shown that, according to his views, the deflection of the ring 28 ft. above the base was considerably larger than it would be according to the cylinder theory; therefore, nothing less than the water pressure should act at that elevation. Even if by a more judicious selection of the modulus of elasticity, a larger deflection might be calculated by the cylinder theory, the actual deflection is so large that no reputable engineer will dare to be satisfied unless the arch ring is strong enough to sustain the entire water pressure. What, according to the author, happens in such a case? The negative thrust or pull (the author seems to speak of a positive thrust), due to the shortening of the arch, according to his Equation (1*a*), is:

$$H = 0.75\, f'_c \frac{t^3}{h^3}$$

Mr. Jorgensen gives*:

$$f_c' = 300 \text{ lb. per sq. in.};$$
$$t = 36 \text{ ft.; and}$$
$$h = 28 \text{ ft., approximately.}$$

Therefore,

$$H = 0.75 \times 144 \times 300 \times \frac{36^3}{28^2} = 2\,000\,000 \text{ lb. per lin. ft.}$$

This pull produces at the abutment a moment of

$$2\,000\,000 \times \left(\frac{2}{3} \times 28 \text{ ft.}\right) = 37.3 \times 10^6 \text{ ft-lb.}$$

According to the author, the extreme fiber stresses equal

$$\frac{37.3 \times 10^6}{\frac{36^2}{6}} = \pm 172\,000 \text{ lb. per sq. ft.}$$

The pull of 2 000 000 lb. produces, according to Mr. Noetzli's reasoning, a tension of

$$\frac{2\,000\,000}{36} = \ldots\ldots - 55\,500 \text{ lb. per sq. ft.}$$

or a maximum tension of 227 500 lb. and a compression of 116 500 lb. per sq. ft. To these stresses are to be added the compressive stresses from the pure arch action of $300 \times 144 = 43\,200$ lb. per sq. ft., giving a maximum tension of 184 300 lb. and a maximum compression of 159 700 lb. per sq. ft. Evidently, according to the author's approximate theory, the arch ring cannot carry the load; still, the dam has not shown any sign of failure.

William Cain,† M. Am. Soc. C. E. (by letter).—With regard to the Barren Jack and the Salmon Creek Dams, the author raises the interesting question as to whether they can be regarded as fixed at the base or simply supported there. The bending moment on the supposed vertical cantilever at

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), Plate XIII, p. 709.

† Prof. of Math., Univ. of North Carolina, Chapel Hill, N. C.

the crown, 1 ft. thick, the weight of the cantilever being neglected, has not been computed for either dam. It has been computed for the Wooling Dam and, at the base, for full water load, the bending moment* is: $M_w = 220\,900$ ft-lb., and for a fall of temperature of 20° Fahr., $M_t = 127\,700$ ft-lb.

The Wooling Dam is 33 ft. high, vertical on the reservoir side, 2.2 ft. thick at the crest, and 4.4 ft. thick at the base, and will be supposed not to be reinforced. By the usual formula, $M = S \frac{I}{c}$, the unit tension on the up-stream side of the cantilever at the base, due to M_w, is 475 lb. per sq. in., and that due to $M_w + M_t$, combined, is 708 lb. per sq. in. The weight of the dam, 16 335 lb., acts vertically; the line of action cutting the base 0.49 ft. up stream from the center, gives a compression of 43 lb. per sq. in. at the up-stream side. Therefore, the total unit stress, due to the bending moments and the weight of the dam, is: Water load and weight of dam, 432 lb. per sq. in.; and water load, fall of temperature of 20° Fahr., and weight of dam, 665 lb. per sq. in.

For either case, plain concrete would crack and the dam could only be regarded as supported at the base with no resisting moment there. At first, when the water is sufficiently low and the temperature is near the mean, the bending moments could be resisted by the base and the cantilever would be fixed at the base, with a vertical generator. Should the water gradually rise, or the temperature gradually fall, a stage would be reached at which the tension on the up-stream side of the cantilever would exceed the limit for concrete in tension (100 to 200 lb. per sq. in.), and the concrete would crack. The cracking would continue as the water rose to the crest and the temperature fell 20° Fahr., and the dam would change from the condition of fixed at the base, to that of simply supported at the base, with no bending moment there and with a generator inclined down stream for reasons to be given later. As the dimensions of the Barren Jack Dam do not differ greatly from those of the Wooling Dam, the conditions at the base are practically the same, or the dam is simply supported at the base.

The solution of the Wooling Dam, not reinforced, regarded as simply supported at the base, with no bending moment there, was given by the writer,† using 7-place logarithms. In the course of the elimination, too few significant figures were left in certain cases to determine accurately the quantities, particularly the deflections at depths from 22 to 29.3 ft. below the crest. On that account, the solution was repeated, using 8-place logarithms with more satisfactory results. The results are recorded in Table 1, in which,

u = radial deflection at the crown; down stream +, up stream —;
T = tangential stress in the horizontal arch; compression +, tension —;
M = moment in vertical cantilever, reckoned as positive when the bending tends to make the generators of the upper cylindrical surface concave to the axis.

The graphs of the quantities u, T, and M, are given in Figs. 6 and 7, the first referring to the water load, the second to the rise in temperature. The vertical

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2068, Table 4, and p. 2075.
† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 87–91.

line represents the vertical face of the dam with the up-stream side on the right. The scale of the M-curve in Fig. 7 is greater than that of the M-curve in Fig. 6.

TABLE 1.—WOOLING DAM, SIMPLY SUPPORTED AT THE BASE.

d, depth, in feet, below crest.	WATER LOAD, RESERVOIR FULL:			RISE OF TEMPERATURE OF 20° FAHR., RESERVOIR EMPTY:		
	u, in inches.	T, in pounds per foot height.	M, in foot-pounds per foot width.	u, in inches.	T, in pounds per foot height.	M, in foot-pounds per foot width.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	+ 0.362	+ 94 600	— 300	— 0.270	— 23 170	0
3.6	+ 0.342	+ 99 100	— 4 300	— 0.245	— 18 680	+ 1 150
7.3	+ 0.319	+ 101 959	— 15 300	— 0.220	— 12 640	+ 4 100
11.0	+ 0.293	+ 101 900	— 30 300	— 0.194	— 4 850	+ 8 280
14.6	+ 0.261	+ 98 300	— 46 200	— 0.166	+ 5 160	+ 12 940
18.3	+ 0.221	+ 89 700	— 59 400	— 0.136	+ 17 490	+ 17 080
22.0	+ 0.171	+ 74 400	— 66 500	— 0.104	+ 32 710	+ 19 330
25.6	+ 0.121	+ 56 048	— 62 100	— 0.070	+ 50 260	+ 18 500
29.3	+ 0.062	+ 30 566	— 41 900	— 0.035	+ 70 690	+ 12 600
33.0	0.000	0	0	0.000	+ 92 880	0

The deflection curve for water load makes an angle with the vertical at the base, the tangent of which is $\frac{0.062}{3.7 \times 12} = 0.001396$, the angle being 4′ 48″. For the temperature curve, the corresponding angle is 2′ 43″. The influence of the weight of the cantilever has not been included in Table 1. Also, since $M = 0$ at the base, there can be no vertical pressures on the base due either to water pressure or temperature change. The inclinations of the deflection curves to the vertical at the base are evidently due to the difference in the arch thrusts, at and just above the base, corresponding to different deflections. Thus, at the base, $T = 0$, and there is no displacement; whereas, at $d = 29.3$, the compression for water load, $T = 30\,566$ lb. The consequent shortening of the horizontal arch there gives a displacement down stream.

This is the full and complete explanation of the angles in question for the Wooling Dam, and the writer believes that it applies equally to the Barren Jack and Salmon Creek Dams. The case is one of shear (as in the case of torsion), in which each horizontal arch of infinitesimal height, slips over the one just below it.

This reasoning does not explain the "knees" in the deflection lines of the Salmon Creek Dam, since for a homogeneous dam, the deflections should continue to increase to the top (Figs. 6 and 7). The material was evidently not of uniform quality there, and it looks reasonable to suppose a weakening of the cantilever and a possible crack.

Attention may be called to the crossing of the deflection curves of May 25th and August 10th, 1909, of the Barren Jack Dam. Of course, the first curve should lie entirely to the right of the second, as the temperature is lower by 13° Fahr. The explanation appears to be either that the observations were inexact, or that the lower half of the dam did not respond as quickly as the

upper part to the change in air temperature. The reservoir was full on both dates and the change in deflection at the crest was $0.58 - 0.52 = 0.06$ in., or 0.00462 in. per degree.

Compare this with the results for December 24th and December 26th, 1908, when, with the reservoir empty, there was a fall in temperature of 30° Fahr. and a deflection change of 0.052 in., or only 0.0017 in. per degree, less than half the preceding rate. It should have been greater than the preceding rate, for many of the arches were in tension and doubtless cracks were in evidence both in the body of the dam and at the abutments, as there was no reinforcement. Evidently, in the two days, from December 24th to December 26th, the time was not sufficient for the dam to respond to the change in air temperature of 30° Fahr. There was a lag, and this should always be considered for cases of sudden changes of temperature.

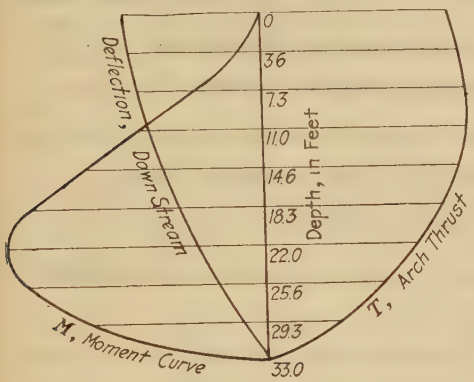


FIG. 6.

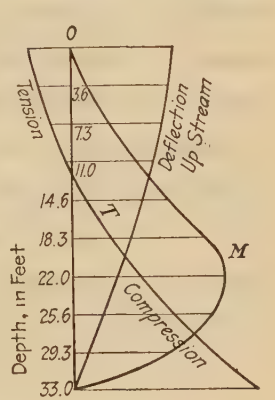


FIG. 7.

In fact, there will always be a lag, even with slow changes in air temperature; and, during such periods, observations of the air thermometer should be recorded daily in order to make a reasonable estimate of dam temperature at a certain date; otherwise, the observations may prove worthless for computing stresses from deflections. For the Barren Jack Dam it is evident that the observations are not full and precise enough to give confidence in the results of computations of stresses for temperature changes only. For thick dams, the subject is further complicated in that the temperatures in the body of the dam vary considerably for the same date, although the analysis, leading to definite formulas, has been based on a constant temperature for the whole body of the dam at a given date. Then, again, in all cases, the uncertainty as to the modulus of elasticity of the concrete in the dam, suggests caution in stating final results.

In the derivation of Equations (1) to (9), the author substituted a parabolic arch for the circular one, replaced the arc length by the span length, and, for the abutment, used the crown thrust for the tangential component. Such approximations may lead to practical results for central angles from about 0 to 30°, for thin arches; but for larger central angles, up to 180°, particularly for thick arches, the results may be greatly in error.

The writer has developed a nearly exact theory of "The Circular Arch under Normal Loads"*, which omits only the influence of shear on deformation, which paper, for brevity, will hereafter be referred to as C, with the formula or table referred to added. The notation given in that paper will be used throughout. A comparison of numerical results, for the same arch, by the use of the writer's formulas and tables and the author's computations, should prove instructive. To that end, consider the observations on the Salmon Creek Dam of October 8th, 1914, and October 27th, 1915. Since the temperatures are not given, they necessarily will have to be assumed to be the same. The deflection at Elevation 1 115, or 108 ft. above the base, on October 8th, 1914, was zero, and on October 27th, 1915, with the water reaching the crest, it was 0.82 in., which will be attributed entirely to the water load. Mr. Noetzli gives the following data: $E = 2\,500\,000 \times 144 = 360\,000\,000$ lb. per sq. ft.; $r = 282$ ft., $h = 76$ ft., and $t = 24$ ft. The central angle, consequently is $2\,\phi_1 = 86° 10'$. Here, $\frac{t}{r} = 0.085$, and the deflection is $\eta = \frac{0.82}{12} = 0.0683$ ft. In the formulas, all dimensions are given in feet.

From C Table 1, for "fixed ends," $c = 1.74$ in the formula, $\eta = c\frac{p\,r^2}{E\,t}$. On solving this formula for $(p\,r)$ and substituting numerical values:

$$(p\,r) = \frac{\eta}{c}\frac{E\,t}{r} = 1\,150\,000 \text{ lb.}$$

Therefore, $p = 4\,080$ lb. per sq. ft., and the water pressure on the extrados of the horizontal arch, 1 ft. in depth, is:

$$p' = p\frac{r}{r'} = 4\,080\,\frac{282}{294} = 3\,913 \text{ lb. per sq. ft.}$$

The moment at the crown of the horizontal arch, at Elevation 1115, is given by the formula,

$$M_0 = -a\frac{E\,t^3}{r\,h}\eta\text{†}.$$

The coefficient, a, was computed, but the result, $a = 0.152$, was found to be the same as that obtained from C Table 3. Substituting numerical values,

$$M_0 = -2\,435\,000 \text{ ft-lb.}$$

corresponding to compression at the extrados and tension at the intrados.

The thrust, P_0, at the crown is to be derived by aid of C Equation (12),

$$(p\,r - P_0) = \frac{-M_0}{r\left(1 - \frac{\sin\phi_1}{\phi_1}\right)} = \frac{2\,435\,000}{282 \times 0.0916} = 94\,400$$

Therefore,

$$P_0 = (p\,r) - 94\,400 = 1\,055\,600 \text{ lb.}$$

By the well known formula:

$$s = \left(\frac{P}{t} \pm \frac{6\,M}{t^2}\right) \div 144,$$

* See p. 233.

† η is to be taken positive down stream, negative up stream.

the unit stresses, in pounds per square inch, at the crown, where $P = P_o$, $M = M_0$, are found to be:

At the extrados, $s = 468$ lb. per sq. in. (compression)
At the intrados, $s = 126$ lb. per sq. in. (compression)

Mr. Noetzli's figures are 461 and 91 lb. per sq. in., respectively.

At the abutment of this horizontal circular arch (of 1 ft. depth), the tangential component of the thrust, P_1, and the moment, M_1, must be computed from C Equation (4) and C Equation (3), or C Equation (13).

$$P_1 = (p\,r) - (p\,r - P_0)\cos\phi_1 = 1\,081\,000 \text{ lb.}$$
$$M_1 = M_0 + (p\,r - P_0)\,h = + 4\,739\,000 \text{ ft-lb.}$$

Therefore, by the formula for s, putting $P = P_1 = 1\,081\,000$, $M = M_1 = 4\,739\,000$ and $t = 24$, we find:

At the intrados, $s = 656$ lb. per sq. in. (compression)
At the extrados, $s = -22$ lb. per sq. in. (tension)

The author's figures are, + 645 and — 89 lb. per sq. in., respectively.

At the crown, the center of pressure is $\frac{M_0}{P_0} = -2.31$ ft. from the center of the crown joint, on the extrados side. The center of pressure then gradually approaches the center line of the arch ring, and crosses it where $M = 0$, or by C Equation (13), where $\cos\phi = \frac{\sin\phi_1}{\phi_1}$, or at $\phi = 24° 43'$. Below this point, the center of pressure recedes from the center line of the arch ring, gradually increasing its distance from it, until, at the abutment, the maximum departure from it, $\frac{M_1}{P_1} = + 4.39$ ft., is reached, on the intrados side.

The author gives, on page 300, the dimensions for the highest arch of the Salmon Creek Dam. The deflection is 0.98 in. for a full reservoir. Following the previous method, we find,

$$(p\,r) = 287\,200 \text{ lb.}$$
$$p = 884 \text{ lb. per sq. ft.}; \quad p' = 850 \text{ lb. per sq. ft.}$$
$$M_0 = -18\,000 \text{ ft-lb.}; \quad P_0 = 287\,000 \text{ lb.}$$

At the extrados, $s = 353$ lb. per sq. in. (compression)
At the intrados, $s = 311$ lb. per sq. in. (compression)

The author does not attempt to estimate the part of the water load carried by this arch; therefore, no final figures are given.

These solutions by the writer are complete, since they include the effects of both bending moment and axial stress, so that nothing has to be added for axial stress due to water load.

When the observed deflection of an arched dam, at any point, is due partly to water load and partly to temperature change, the solution is more difficult. It will be found that the value of $(p\,r)$ can be determined accurately, but that the moments and stresses can be ascertained only approximately. The solu-

tion is based on the reasoning given by the writer in a previous discussion*, leading to the formula for deflection, D,

$$D = \frac{3}{2}\frac{r'\,T}{E\,t} - e\,r'\,t_0,$$

in which the Smith formula for deflection, due to T, is given, in order, presently, to compare numerical results.

In this case,

T = tangential stress, for width, t, at crown of horizontal arch of depth 1 ft., due both to water load and temperature change. When in compression, it is positive; when in tension, it is negative.

r' = radius of extrados (= 135 ft. for Wooling Dam).

e = coefficient of expansion for change of temperature of 1° Fahr. = 0.000 0055 (Smith).

t_0 = number of degrees Fahrenheit of change in temperature above or below an assumed mean. For rise of temperature, t_0 is positive; for a fall, it is negative.

If the tangential stress due to water load, is denominated as T_w, and that due to change of temperature, T_0, then T_w is always positive, but T_0 can be positive or negative. Also, since t_0 can be positive or negative, there are four cases, all of which were carefully provided for in the foregoing formula. Careful attention must be given to the algebraic signs. When D is positive, it indicates a radial deflection down stream.

As Smith gives the modulus for the horizontal arch,

$$E = 2\,000\,000 \times 144 \text{ lb. per sq. ft.},$$

it will be used in this formula. For a rise of temperature of 20° Fahr., on substituting numerical values and multiplying by 12 to reduce to inches,

$$D = 0.000\,00844\,\frac{T}{t} - 0.178$$

in which D is in inches. For a fall in temperature of 20° Fahr., change the sign of 0.178. T and t have the same values as before, t being in feet, and T in pounds.

In Table 2, Columns 5 and 8 are computed by the foregoing formulas. The other columns are filled out from Table 1 for the Wooling Dam, not fixed at the base. For t_0 positive, the quantities for water load and temperature are algebraically added. Thus, the total deflection and thrust are due to both causes. For t_0 negative, change the signs of u and T, under "Temperature changes", and add algebraically to the values of u and T, respectively, in the second and third columns of Table 1.

As the third decimal figure in the values of u in Table 1 cannot be relied on, the results for deflections (Columns 3 and 5 or Columns 6 and 8) compare very well. Similar results follow from use of the quantities pertaining to the dam fixed at the base. This should tend to give confidence in the formula for D and its applications.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 79–87.

A more accurate formula for D will now be written, by replacing the first term for deflection, due to T, by $c \frac{p\, r^2}{E\, t}$, and the last term by, $e\, r\, t_0$,

$$D = c \frac{p\, r^2}{E\, t} - e\, r\, t_0$$

in which D is in feet, and, as before, r is the radius of the center line of an arch, on which the normal unit pressure, due to the combined water load and temperature change, p, is positive when directed toward the center; otherwise, it is negative. For a given central angle and a value of $\frac{t}{r}$, the value of c is to be taken from C Table 1.

TABLE 2.

		Water Load and 20° Fahr. Rise of Temperature.			Water Load and 20° Fahr. Fall of Temperature.		
d, in feet.	t, in feet.	Total deflection, in inches.	Total thrust, T, in pounds.	D, in inches.	Total deflection, in inches.	Total thrust, T, in pounds.	D, in inches.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	2.2	+ 0.092	+ 71 430	+ 0.087	+ 0.632	+ 117 770	+ 0.630
18.3	3.42	+ 0.085	+ 107 190	+ 0.096	+ 0.357	+ 72 210	+ 0.356

On solving for $(p\, r)$, we find,

$$(p\, r) = \frac{E\, t}{c\, r} (D + e\, r\, t_0),$$

all dimensions being in feet. For a water load and rise of temperature, t_0 is positive; for a water load and fall of temperature, t_0 is negative.

Some of the results for the Barren Jack Dam will now be examined. On December 24th, 1908, with reservoir empty, the temperature was 87° Fahr.; and the deflection was zero. On August 10th, 1909, with reservoir full, the temperature was 60° Fahr., and the deflection, at a point 17 ft. above the base, was 0.36 in. In this case, there was a fall in temperature of 27° Fahr., thus

$$D = \frac{0.36}{12} = 0.03 \text{ ft.}$$

$$t_0 = -27$$

At the point 17 ft. above the base, the author gives: $r = 78$ ft., $h = 10$ ft., $t = 4$ ft.; whence, $2\, \phi_1 = 58° 40'$, $\frac{t}{r} = 0.0513$, and, therefore (C Table 1), $c = 1.65$, for an arch with "fixed ends".

Let $E = 2\,500\,000 \times 144 = 360\,000\,000$ lb. per sq. ft.; $e = 0.0000055$; therefore, $r\, e\, t_0 = -0.01158$, and $(D + e\, r\, t_0) = 0.01842$. Substituting numerical values in the formula,

$$(p\, r) = 206\,100 \text{ lb.}$$

The deflections under the water load and a fall of temperature are both down stream, giving compression at the extrados, tension at the intrados, from bending moment at the crown, which is negative.

From C Table 3,

$$a = 0.160 \text{ for water load;}$$
$$a = 0.170 \text{ for temperature.}$$

Let,

η_w = deflection from water load,
η_0 = deflection from fall of temperature.

Both these are unknown, but the following relation is true:

$$\eta_w + \eta_0 = D.$$

The total moment at the crown from both deflections, can be written:

$$M_0 = -\frac{E\,t^3}{r\,h}\,[0.160\,\eta_w + 0.170\,\eta_0]$$

An assumption will have to be made as to the values of η_w and η_0, expressed in terms of D, in order to effect a solution. Fortunately, the Barren Jack Dam and the Wooling Dam have nearly the same dimensions, and by reference to Table 1, it may be inferred that the deflections for the Wooling Dam, from water load and a fall of temperature, of 27° Fahr., are nearly the same; so that it will be assumed that the same relation holds for the Barren Jack Dam. Assume that, $\eta_w = \eta_0 = 0.5\ D$, therefore,

$$[0.160\,\eta_w + 0.170\,\eta_0] = \frac{1}{2}\,[0.160 + 0.170]\,D = 0.165\,D.*$$

On substituting numerical values,

$$M_0 = -\frac{360\,000\,000 \times 64 \times 0.165 \times 0.03}{78 \times 10} = -146\,200 \text{ ft-lb.},$$

$$\therefore p\,r - P_0 = \frac{-M_0}{r\left(1 - \dfrac{\sin\phi_1}{\phi_1}\right)} = \frac{146\,200}{3.354} = 40\,600 \text{ lb.}$$

$$\therefore P_0 = p\,r - 40\,600 = 165\,500 \text{ lb.},$$

and the unit stresses at the crown are,

$$s = \frac{1}{144}\left[\frac{165\,500}{4} \pm \frac{6 \times 146\,200}{16}\right]$$

$$= \begin{cases} + 668 \text{ lb. per sq. in. (compression) at the extrados;} \\ - \ 93 \text{ lb. per sq. in. (tension) at the intrados.} \end{cases}$$

The author derives,

$$\begin{cases} + 495 \text{ lb. per sq. in. (compression) at the extrados;} \\ - 242 \text{ lb. per sq. in. (tension) at the intrados.} \end{cases}$$

As another illustration of the decided influence of temperature change compare the deflection lines, Fig. 1, for December 24th, 1908, and for May 25th, 1909, at a point 17 ft. above the base. The fall of temperature is 87 — 47 = 40° Fahr.; and the deflection is, 0.32 in. Therefore, $t_0 = -40$, and $D = \dfrac{0.32}{12} = 0.0267$ ft. Following exactly the method used in the last

*If, for a much greater fall of temperature, it was estimated that $\eta_w = 0.4\ D$; then, since, $\eta_w + \eta_0 = D$, it follows that $\eta_0 = 0.6\ D$, and $[0.160 \times 0.4 + 0.170 \times 0.6]\ D = 0.166\ D$. This illustrates the method to be followed in any case.

example, writing $(0.160\ \eta_w + 0.170\ \eta_0) = 0.167\ D$, the unit stresses are found to be, at the crown,

$$\begin{cases} + 460 \text{ lb. per sq. in. (compression) at the extrados;} \\ - 226 \text{ lb. per sq. in. (tension) at the intrados.} \end{cases}$$

In this case, the thrust and bending moment at the crown are $P_0 = 67\,300$ lb. and $M_0 = -131\,600$, respectively. The great decrease in the thrust and a smaller (numerical) decrease in the bending moment, as compared with previous values, accounts for the great differences in unit stresses. Note that the unit tension at the intrados (226 lb. per sq. in.) is much larger than the value (93 lb. per sq. in.), as computed for the preceding case.

The horizontal arch considered, was taken as "fixed at the ends", in order to compare the results with those of the author. As this arch was only 4 ft. thick, and not tied at the ends with reinforcement, it is doubtless intermediate between an arch "fixed" and hinged at the ends; so that, properly, the solution should be made for both cases, and intermediate values for the unit stresses written, depending on the conditions at the ends as constructed.

As to the confidence to be placed in results, it may be noted that the theory for water load alone is exact, but the theory of the influence of temperature changes is approximate. As first outlined by B. A. Smith, M. Am. Soc. C. E.,* the latter theory is open to some objections; but as it is the best that has been proposed, this theory was adopted; particularly as it was believed that it gave close approximations to temperature stresses in arched dams. There is another matter of importance which affects the accuracy of computed stresses from observed deflections, namely, the influence of vertical cracks in the masonry. Such cracks must close before arch action takes place, and there is a deflection during this closing when cantilever action alone is exerted. The total observed deflection is made up of the part due to the closing of the cracks and the part corresponding to the dam exerting both full arch and cantilever action. The formulas do not take account of the deflection due to closing of the cracks, and not completely of the subsequent deflection, since the dam is under action as a cantilever at the beginning of this later deflection. If the total deflection is used in the formulas, the dam is assumed to be without cracks at any stage. If it was possible to observe the deflection and temperature at the time of closing of any cracks, then the subsequent deflection can be regarded as the value of D in the formula, but t_0 must be taken as the difference between the temperatures (with the proper sign) at the time of closing of the cracks and the time of the final observation when the deflection is greatest. If the total deflection, including that corresponding to the closing of the vertical cracks, is taken as the value of η in the formula,

$$(p\ r) = \frac{\eta}{c}\frac{E\ t}{r}$$

the value of $(p\ r)$ will always be too great. In fact, for quite large cracks, it is easy to conceive of the deflection due to the closing of the cracks being the greater part of the total deflection, so that $(p\ r)$, as computed, should have more than double its true value. Of course, if $(p\ r)$, as computed, should prove

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

to be greater than its value for full water pressure at the base of the dam, the absurd result would show at once, either the influence of wide cracks or inexact observations.

It is thus seen that the author's analysis, in addition to the approximations introduced by substituting a flat parabolic arc for the circular arc, suffers from the lack of a precise method for estimating the part of the water load sustained by the arch. At the crest of the dam, it is never attempted. However, it is easy to supply this omission, since $(p\ r)$ can be readily computed, as already shown, either by the use of the author's formula for deflection or by the writer's more precise formulas for either fixed or hinged ends.

The arch thrust due to rib-shortening alone is given in a precise manner by the author's Equation (14), in connection with Fig. 4. Similarly, the author's Equation (18), in connection with Fig. 5, gives a precise estimate of the thrust due to temperature changes. Many of the results of C Table 2, expressed in terms of $\frac{t}{h}$, were compared with those of the author for temperature thrust and found to agree very well, thus affording a mutual check. It must be borne in mind that such values of H_t pertain to a free arch and do not include the influence of the reactions of the cantilevers. Both influences must be included to find the thrust for the arch regarded as a part of the dam.

In conclusion, it may be stated that the author has done good service in calling attention to the importance of a proper interpretation of the observations of deflections of curved dams. Such observations should be made with extreme care, or they may be misleading, remembering that minute changes in deflection correspond to appreciable changes in stresses. Further, daily observations of the temperature of the air should be recorded, and the times for observations of deflections should be at periods when there is not much change in temperature.

B. F. Jakobsen,* Assoc. M. Am. Soc. C. E. (by letter).—Referring to the discussion of the deflection curve of the Salmon Creek Dam, the author states:

"A cantilever fixed at the base * * * would have the line of zero deflection as a tangent, approximately as shown by a typical cantilever deflection curve in Fig. 2."

As the writer showed† in discussing the author's paper, "Gravity and Arch Action in Curved Dams," the deflection curve of a dam like the Salmon Creek Dam is not vertical at the base, but makes a considerable angle with the vertical due to the shear deformation. The author's statement is approximately true for ordinary beams, in which the length of the beam is considerable compared with its height; when the length of the beam is considerably less than its height, the deflection due to shear exceeds the deflection due to bending, as shown in Table 15 and Fig. 32, of the writer's discussion referred to.

* Designing Engr., San Joaquin Light and Power Corporation, Fresno, Calif.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 99.

The angle, α, at the base of the Salmon Creek Dam, on the assumption that the dam acts as a cantilever, is found from the writer's Equation (45)*:

$$t g \alpha = \frac{d y_h}{d x} = \frac{1.2 S_h}{144 G b}$$

in which $S_h = 31.3$; $h^2 = 31.3 (H - x)^2$ lb. for 1 ft. width of dam; G is the coefficient of rigidity $= \frac{E}{2.4}$ 1 040 000 lb. per sq. in., b = the base = 42 ft. and $H = 160$ ft.†

This gives $\alpha = 0° 0' 33''$, at the base. The typical cantilever deflection curve, as shown by the author in Fig. 2, is given in the correct shape in Fig. 32 of the writer's discussion to which reference has been made.

The lower 28 ft. of the dam may be considered as a rectangular section, the base being 42 ft. and the height 28 ft., as shown in Fig. 3. The top of this 28-ft. section has moved down stream § in., and this is accounted for by the author by cracks which he claims have been produced in the up-stream face of the concrete and close to bed-rock. He then says:

"Such cracks, of course, will extend only for a comparatively short distance into the dam body, and the water-tightness is insured sufficiently by the increased compression stresses near the down-stream side."

If the dam has developed cracks in a horizontal plane near the base, then the total deformation of the concrete is practically negligible and that means that the crack in the up-stream face must be:

$$0.375 \times \frac{42}{28} = 0.56 \text{ in.}$$

If this crack exists it must extend practically through the dam, and seepage would most likely take place at this point. The probability would be in favor of one large crack instead of several small cracks.

To account for the §-in. movement 28 ft. above the base of the dam, the author calculates that a uniformly distributed load of 20 000 000 lb. would be required, and, in doing so, entirely neglects the shear deformation. The shear diagram is a triangle 28 ft. high with 25 400 000 lb.‡ as a base and, therefore,

$$S_h = 25\,400\,000 \frac{H - x}{H}$$

and the deflection due to shear alone is,

$$y'_h = \int_{x=0}^{x=28} \frac{12 \times 1.2}{144 G \times b} \times S_h \, d x = 0.815 \text{ in.}$$

The total deflection due to shear and bending is, therefore, 1.19 in., or 3.17 times as great as the deflection calculated by the author. The load distribu-

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 99.

† *Loc. cit.*, Fig. 31, p. 99.

‡ Using $E = 2\,500\,000$, as the author does, the writer obtains 25 400 000 lb. instead of the author's 20 000 000 lb. and a tension of 8 400 lb. per sq. in., instead of the author's 6 600 lb. per sq. in.

tion assumed by Mr. Noetzli seems very improbable. The more proper load distribution would be to assume a shearing force as a maximum 28 ft. from the base and as zero at the base, so that,

$$S_h = 25\,400\,000\,\frac{x}{H}$$

This gives a deflection 6.34 times as great as that computed by Mr. Noetzli and, therefore, a maximum stress of 1 325 lb. per sq. in. instead of the author's 8 400 lb. per sq. in. This is on the assumption that the base is fixed, which, in the writer's opinion, is untenable.

The author has evidently overlooked the most interesting point on the deflection curve, that is, the knee at Elevation 1075 (Fig. 2). This point is deflected 0.625 in., while the point at Elevation 1095 is deflected only 0.540 in., or 0.085 in. less. If the fact that the dam below Elevation 1075 is not vertical is taken into account, the relative deflection is 0.185 in. in the 20 ft. of dam. The average thickness of the dam at this place is approximately 31 ft., so that if this deflection is due to bending with a uniform load, as the author assumes, there must exist in the down-stream face, at Elevation 1075, a vertical tensile stress of:

$$s = \frac{2\,E\,h}{L^2}\,y = 5\,950 \text{ lb. per sq. in.}$$

in which,

$E = 2\,500\,000$ lb. per sq. in.;
$h = 31 \times 12$ in.;
$L = 240$ in.; and
$y = 0.185$ in.

If it is assumed, as seems more logical, that the beam is 40 ft. long, fixed at the ends, and uniformly loaded, and that the deflection in the middle is 0.085 in., then,

$$s = \frac{16\,E\,h}{2\,L^2}\,y = 2\,750 \text{ lb. per sq. in.}$$

There should be, therefore, according to the author's theory, a horizontal crack in the down-stream face at Elevation 1075. As no crack has been reported, the writer assumes that none exists.

The fact that no horizontal crack has developed in the down-stream face at Elevation 1075 affords an excellent example of the large deformation a concrete dam can undergo, without producing cracks, and in spite of the fact that a tension of about 2 750 lb. per sq. in. should exist at that point according to the ordinary formula for bending. It shows that the ordinary bending formulas are entirely inapplicable.

In view of the fact that no cracks have developed in the visible down-stream face, there is no reason to expect any cracks in the up-stream face. It is very fortunate that Mr. Noetzli has applied his theory to the Salmon Creek Dam, as this affords a means of checking his analysis against known facts and shows that his assumptions of cracks in the up-stream face are not verified.

Regarding the accuracy of the deflection measurements, L. R. Jorgensen, M. Am. Soc. C. E.,* mentions the great difficulty of measuring such deflections accurately near the base of the dam, and any one familiar with such measurements will not fail to realize this.

The aim of the author, to determine stresses from deflections, is far too ambitious an undertaking with the present state of knowledge. The method may yield valuable and approximate information in the hands of one who fully understands the limitations imposed by the assumptions. In unskillful hands, the method is bound to lead to conclusions which cannot be substantiated and which are likely to be incorrect.

In the interest of scientific precision, the writer would suggest that phrases such as that used by the author, for example:

"A further proof that this dam has developed horizontal cracks at the up-stream side is shown by the following calculations: * * *"

should not be used, unless the dam is known to have cracked. As expressed by the author, it may be misleading.

In summing up, the writer can do no better in expressing his opinion than by quoting from the discussion, by D. C. Henny, M. Am. Soc. C. E., of the author's paper, "Gravity and Arch Action in Curved Dams":†

"The reason that the mathematical treatment by the author carries no conviction is that, owing to the complexity of the problem, it is extremely incomplete and ignores numerous important elements to which he himself, * * *, makes able reference."

CHARLES P. DUNN,‡ ASSOC. M. AM. SOC. C. E. (by letter).—An appreciation of Mr. Noetzli's enthusiasm and his able efforts to solve a difficult problem, prompts the writer to add to the discussion, in the hope of being helpful.

The large amount of capital to be spent in projected water-control works in the near future makes it desirable that the study of arch dams should be continued, in order that engineers may accept improved methods of design as soon as possible. Even a slightly better understanding of the subject will save many thousands of dollars.

On account of the shrinkage of concrete on setting, and the variations of volume with temperature, it appears to be practically impossible to design an unreinforced arch dam in which the cantilever stresses will not be excessive (at least from a theoretical standpoint), unless the deflection allowed by shrinkage is largely eliminated by grouting the cracks and construction joints or by closing narrow vertical openings in the dam when the temperature is slightly below the mean after shrinkage in the main body of concrete has taken place.

The writer favors the latter method because, as Mr. Noetzli has pointed out, if the grouting is poorly done, the dam is injured by an uneven distribution of compressive stresses and the introduction of bending moments in the arches. It has also been pointed out that it is impossible to know positively to what extent a joint or crack has been filled with grout, and attention has been

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 319.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 121.

‡ Designing Engr., Skagit River Power, Seattle, Wash.

called to the difficulty and uncertainty of building up a high grouting pressure, due to the friction head of grout flowing in small cracks, and to the difficulty of forcing the crest up stream, due to the leakage of grout from the last seam to be filled when sufficient pressure is applied to move the crest.

No agreement has been reached on the amount and importance of rock deformation under and at the abutments of the dam. These deformations are classed by different engineers from infinitesimal and negligible to appreciable and of consequence. It appears that unless deflection measurements are complete, covering practically the entire surface of the dam, an unknown rock deformation would introduce error into the determination of stresses from deflections.

If complete analysis of stresses is to be made from deflection measurements, the existence or non-existence of cracks must be ascertained, and existing cracks must be located.

Engineers do not agree as to what extent the uncertain element of the "flow" of concrete or the change of the modulus of elasticity with time may enter into the determination of stresses from deflection measurements.

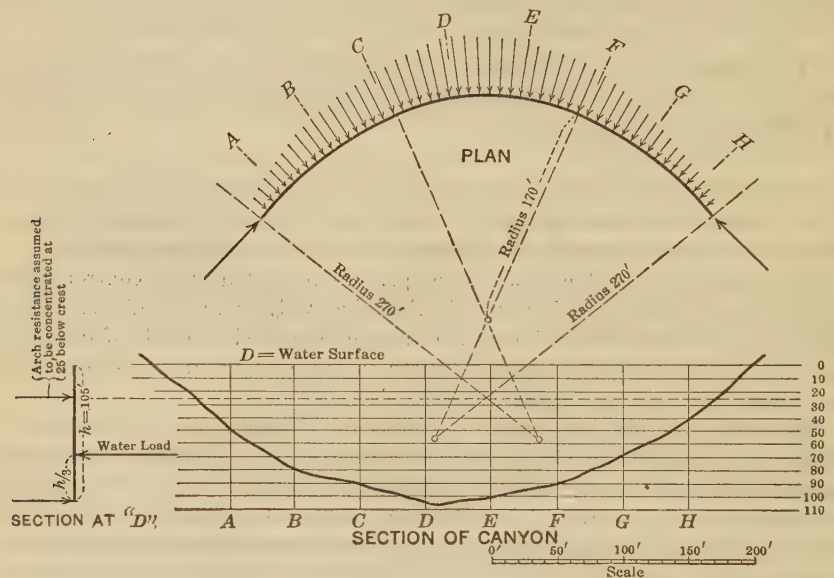


FIG. 8.

Mr. Noetzli has already shown* that in the ordinary type of arch dam the upper horizontal arches carry a greater load than is imposed on them directly by water pressure, the additional load being transferred to the upper zone by vertical beam action. He has worked out a method for determining approximately the distribution of load between the vertical beams and the arches by considering that the beams are supported throughout their length by arches of varying stiffness or ability to carry load. The writer is confused by the thought that,

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

in addition to the foregoing assumption, it seems necessary to consider also that the arches are supported throughout their length by cantilevers of varying stiffness and that load may be transmitted horizontally from a region of flexible cantilevers to a region of stiff cantilevers by a portion of the arch acting as a beam. In other words, if the cracks are closed and the dam is acting as a whole, the deflection of any single point in the dam changes the stress in every horizontal arch and every vertical beam in the dam, which condition throws doubt on the applicability of the comparatively simple formula used by Mr. Noetzli to obtain the arch deflection. It seems that the use of this formula might introduce an error of some magnitude into the calculations for a dam built at an irregular site. In his evidently thorough study of the problem, Mr. Noetzli has no doubt covered this point.

It appears to the writer that, in general, the arches in the usual type of dam have not a uniform normal load, for the following reasons:

1.—The upper arches are loaded by (*a*) a uniform normal load imposed directly by the water pressure, plus (*b*) an additional load imposed by the vertical beams, which additional load is greater near the crown than near the abutments. Mr. Noetzli's calculations show that a certain additional load (*b*) exists. This additional load will probably be greater in quantity than his calculations show, because of rock deformation and because of hydrostatic pressure under the dam. The effect of these uncertain deformations and forces will be to increase the condition of uneven loading of the arches.

2.—Take a zone near the middle of the height of the dam. The arches may carry a certain part of the load near the crown, which loading diminishes to zero as one goes along a horizontal plane to the side of the canyon, at which point all the load is carried by the cantilevers.

To illustrate this point an approximate calculation has been made of a hypothetical dam which is assumed to have no resistance as a cantilever, the water load being carried by vertical beams supported at the base by the foundation and near the top by an arch in a horizontal plane, all the resistance by arch action being assumed to be concentrated at one elevation. The site is the usual U-shaped canyon, as shown in Fig. 8. With this extreme assumption, the uneven loading of the horizontal arch gives a linear arch of maximum curvature at the deepest part of the canyon, the curvature decreasing to almost a straight line at the abutments. The three circular arcs occupy closely the position of the somewhat irregular linear arch.

The true condition in an actual dam must be such that the center line of pressure resulting from the loading of the horizontal arch slices will be somewhere between a circle and the hypothetical extreme shown in Fig. 8.

It appears to the writer that a deviation from the exact circular form, following as closely as practicable the true line of pressure of the elastic arch as determined by studies of existing structures, will effect an economy of material because of reduction of bending moments in the horizontal arches, without necessarily increasing the unit stresses now considered proper.

Such a deviation from the circular form concentrates a larger portion of the total curvature in the deeper part of the canyon, and, at a few sites, might make

it economically advisable to substitute conical surfaces for the warped surfaces of the constant-angle arch.

A test of miniature dams covering this point would be extremely interesting and would probably develop some worth while data.

ALBERT B. HILL,* M. AM. SOC. C. E. (by letter).—The author has raised some interesting questions which merit discussion.

If the deflections of the dam have been carefully measured and the character of the forces that produced the deflections, together with the elastic modulus of the material, are definitely known, the stresses resulting from the deflections should be determinable. There can be no serious question on that point.

In referring to "Stresses in the Vertical Cantilevers of Arch Dams", the author bases his method of determining those stresses on the "well known theory of stresses and corresponding deflections in cantilever beams". The usual formulas for cantilever beams are deduced from the theory of flexure, assuming the beams to be rigidly fixed at the base, with compression on one side of the neutral axis and tension on the other side. The writer questions whether formulas are strictly applicable to dams generally, which may not be rigidly fixed at the base.

The author assumed the dam to be "rigidly fixed at the base due either to its own weight or to special anchorage to the bed-rock". The value of the weight, as a fixing force, is limited to the condition of the dam as simply supported at the base. When the load commences to produce tension in the up-stream face of the dam near the base, the weight of the dam as a fixing force is exhausted. An adequate anchorage for rigidly fixing the base is required, if the formulas for rigidly fixed cantilevers are to apply. The absolute adequacy of such anchorage for rigidly fixing the base permanently, may be uncertain.

In his investigations of the Barren Jack Dam in Australia and the Salmon Creek Dam in Alaska, the author finds that the deflection curves, instead of being tangent to the normal at the base, make material angles with the normal at their intersection with the base. Such a condition indicates clearly, that whatever the intentions of the designers may have been, the dams are not rigidly fixed at the base, and renders questionable the application of formulas for a rigidly fixed base. If the dams are simply supported at the base, the deflections would be greater than if they are considered as rigidly fixed at the base. This may help to account for some of the great differences between the measured deflections and the calculated deflections of the author.

The writer doubts the wisdom of using steel reinforcement for anchoring and for resisting tension in high concrete dams subject to heavy water pressure. It is imperative that important impounding reservoirs should be designed as permanent structures. The dangers to life and property and the loss that would be caused by a failure renders inadmissible the construction of any but permanent dams. The steel reinforcement to be most efficient should be reasonably near the up-stream face of the dam. The farther it is removed from the up-stream face, the more steel is required. The amount required is

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large in any case if the up-stream face of the dam is to be "rigidly fixed" at the base.

Concrete, if of good quality and under only light water pressure, may protect the steel fairly well. Concrete, however, is more or less porous and water, under heavy pressure, will be forced through a few inches or even a few feet of it. If the unit stresses are equal to those generally allowed in steel reinforcement, then, in accordance with the laws of elasticity, the steel will stretch somewhat. This will produce tension in the concrete, tending to open fine hair cracks and make it still more porous.

Water under high pressure is also likely to get to the steel through contraction joints and through construction joints where weak laitance may not have been entirely removed. Damp spots, where water has seeped through, are not infrequently noticed on the down-stream face near the bottom of dams of only moderate height, particularly soon after the dams are put in commission.

If water does get to the steel and corrosion takes place, the concrete is likely to be split and trouble will result where it may not be seen. Corrosion of the steel has already caused trouble in reinforced concrete stand-pipes built less than a score of years ago. The writer believes it is safer to omit the steel and use a little more concrete, and consider the dam as simply supported at the base, unless the steel is much more efficiently protected than in present practice.

A force which is helpful to the arched dam, and which seems to have been given little, if any, attention, is simple vertical beam action. Vertical slices of the dam may be considered as simple vertical beams, supported at the base in the rock and, to a limited extent, supported at the top by horizontal arch action. In considering arch action, the dam is assumed to be divided into horizontal slices free to slide on each other. On this assumption, the horizontal arch slices at the top, theoretically, are not stressed from water pressure, as they are above the water level. These top horizontal arches should be made to do useful work, which they actually do, by acting as limited supports to the top of the simple vertical beams.

The writer believes that the top of an arch dam should be of substantially the same thickness as the top of a corresponding gravity section dam. The rigidity of a simple beam varies as the cube of the depth. If an arch dam is, say, 12 ft. thick at the top and 60 ft. thick at the base, the thickness at the base being five times the thickness of the top, the rigidity near the base would be about one hundred and twenty-five times the rigidity near the top. The maximum water pressure is also at the bottom. Therefore, the simple vertical beam, with relatively small support at the top, is able by its rigid bottom portion, to transmit a material part of the bottom water pressure, into the rock foundation.

The simple vertical beam action would tend to produce tension in the down-stream face of the dam, thereby neutralizing a portion of the compression in the down-stream face resulting from simple gravity action. It would also tend to produce compression in the up-stream face of the dam, helping to neutralize any tension in the up-stream face resulting from gravity action, or

cantilever action, in a reduced gravity section. The simple vertical beam action is relatively about four times more efficient than cantilever action. Such action therefore would be helpful to gravity action and permit more load to be put on gravity action without producing tension in the up-stream face of the dam. It also helps in distributing the water pressure over the horizontal arch slices, tending to make the unit stresses in the arches more nearly uniform, as well as helping to transmit some water pressure quite directly into the rock bottom.

The various forces acting in arch dams make them, as the author has stated, "statically complicated structures", and he evidently appreciates the difficulties of an exact mathematical solution by a general formula that would be of practical value.

In the thicker bottom portions of the arch dam, shearing stresses—mostly horizontal and inclined, but some vertical—are a material factor in transferring the loads into the rock. The concrete near the bottom is so massive that the unit stresses in shear are relatively small, even for transmitting the greater part of the bottom water pressure. The principal resisting forces acting in arch dams are shear—horizontal, vertical, and inclined; simple gravity action; horizontal and inclined arch action; simple vertical beam action; some horizontal beam action; and cantilever action. On the principle of least work, all these different forces do work in proportion to their relative efficiencies or rigidities. With all these resisting forces marshaled, one may perhaps cease to wonder that arch dams have not failed.

Considering the dam as simply supported at the base, the deflection due to gravity action can be determined with a close degree of approximation, starting from the resultant pressure curve constructed by considering the weight of the masonry, the water pressure, and a reasonable allowance for uplift. The proper allowance for uplift depends on the character of the foundation. Some engineers (although probably few) advise allowance for uplift equal to the full static head on the base of the dam, or, in other words, that the dam is floating on water, which is impossible. Other engineers advise an allowance for uplift equal to the full static head at the up-stream edge, decreasing to zero at the down-stream toe of the dam. This is more reasonable; but the writer is of the opinion that it is more than necessary in many cases.

Where the foundation is sound, solid rock, where dense concrete is used, and a thick layer of soft mortar is spread over the surface of the rock before the concrete is placed, an allowance for uplift equal to half the static head at the up-stream edge of the dam, decreasing to zero at the toe, or, better, to the ground-water level at the toe, if the ground-water level is above the toe of the dam, will prove sufficient for full gravity section dams.

In arch dams, horizontal arch action and vertical uplift act at right angles to each other. Therefore, theoretically, uplift does not directly affect horizontal arch action. Practically, however, uplift diminishes the efficiency of gravity action and thereby throws more work on arch action. Owing to the probability of a little tension in the up-stream face of arch dams before much arch action

becomes effective, allowance for uplift in arch dams should be fully as much as for full gravity sections. The engineer must use his judgment as to how much allowance should be made for uplift in any special case.

The writer believes that in the allowance for ice pressure there has been a tendency sometimes to over-estimate its importance. It is a well known principle of physics that pressure produces heat, and the melting of ice by pressure applied on it is one of the familiar demonstrations of the principle. If through a rise in temperature, severe pressure, acting slowly, should be caused by ice, the latent heat developed at the point of contact would tend to soften the ice and relieve the pressure. In special cases, allowance for ice pressure may be desirable, but, in most instances, no special allowance is necessary. Any possible ice pressure will be sufficiently cared for by the usual factor of safety of about two.

Before constructing the resultant pressure curve, a load curve should be assumed for the gravity action of the individual horizontal sections into which the dam is assumed to be divided. The assumed load curve should be such that the moments would show practically no tension in the up-stream face of the dam; and such that the greatest efficiency will result in the resisting forces on the principle of least work. Two or three trials will probably be necessary to obtain the best results.

For convenience in interpreting results, a percentage curve can be used, by constructing, to a convenient scale, a rectangle on the back of the section of the dam, so that with horizontal lines drawn from each of the joints considered, the total length of the line or ordinate will represent 100%, or the total load on each joint. The best percentage load curve for gravity action will not be a vertical line, but a curved line. The center of pressure for each horizontal section into which the dam is assumed to be divided must be determined, in order to obtain the moments from water pressure. The depth of the center of pressure for each section, from the surface of the water or top of the dam, can be determined by the well known formula,

$$\text{Depth from surface} = \frac{2}{3} \times \frac{d_1^3 - d_2^3}{d_1^2 - d_2^2}$$

in which d_1 = depth from the surface of the water to the top of the section considered, and d_2 = depth from the surface of the water to the bottom of the section considered. By subtracting the result obtained by this formula, from the height of the dam, or from the distance from the base of the dam to the surface of the water, the moment arm for each section, with center of moments at the base of the dam, will be given. These moment arms grading off gradually for equal heights of section, are easily checked by first and second differences—second differences for the upper half of dam, and first differences for the lower half.

It will be convenient to have first, the moments of the water pressure at each section for the total water-pressure load, from which the moments for the percentage load curve easily follow. It will be helpful in checking to remember that with the center of moments at the base of the dam, the moment of water pressure is greatest at the center of the dam—the moments grading off gradually toward the top and equally from the center toward the base.

When the percentage load curve has been adjusted so that the overturning moments do not produce appreciable tension in the up-stream face of the dam and the resultant pressure curve for gravity action has been constructed, the extreme fiber stresses in the down-stream and up-stream faces of the dam at each joint can be determined from the nearest edge distance from where the resultant pressure curve cuts the joint.

The well known general formula for the greatest unit pressure is:

$$p = \frac{2N}{d}\left(2 - \frac{3e}{d}\right)$$

in which

p = the greatest unit pressure;
N = the normal component of all the external forces acting; and
d = the depth of the joint, or thickness of the dam at the joint.

This general formula can be simplified considerably for special cases.

Having found for each section the maximum unit fiber stress, which will be generally at the down-stream face of the dam, the unit stress in the opposite face follows by substituting for e, in the formula, the longest edge distance from where the resultant pressure curve cuts the joint, or from similar triangles, assuming, of course, that the stress varies uniformly across the section. If the resultant pressure curve falls outside the "middle third", the opposite face stress will be tension.

Assuming a suitable elastic modulus for the concrete and considering the bottom section of the dam, the shortening of the down-stream and up-stream faces resulting from the compression can be found in the usual way; or, a little easier, by taking the difference between the compression at the down-stream and up-stream faces of the section, the shortening that would cause deflection at the top joint of the section easily follows. The deflection equals the shortening multiplied by the height of the section divided by the thickness of the base. This deflection of the bottom section only can be extended by simple proportion up through the different sections and to the top of the dam.

Proceeding to the second section from the bottom, the deflection of this section from the normal to the bottom section can be determined in a similar manner. Similarly, the deflections throughout the height from all the individual sections can be determined. The algebraic sum of all the different sectional deflections will give the total deflection at the different elevations of the gravity section.

If the resultant pressure curve near the top of the dam falls on the up-stream side of the center line of the dam, owing to differences in batter on the up-stream and down-stream faces of the dam, the top sections, where the resultant pressure curve is up-stream from the center line of the dam, will have a slight minus or up-stream deflection. This, however, is greatly overshadowed by the much larger down-stream deflections of the individual sections lower down. So that the resultant deflection curve with reservoir full will be generally down stream from the normal. When the reservoir is empty, there is an initial up-stream deflection to be considered, as the unit stresses

from weight will be greater in the up-stream than in the down-stream face, due to differences in batter in the two faces.

The deflections from arch action must be the same as in the gravity section, because they are in the same structure. Knowing the deflections, the corresponding unit stresses that would produce them can be determined from the deflection formulas of Silas H. Woodard, M. Am. Soc. C. E.,* or B. A. Smith, M. Am. Soc. C. E.,† by transpositions.

The Woodard formula is:

$$D = \frac{l}{2} \cot a,$$

in which,

D = the crown deflection;
l = the shortening of the arch ring under compression; and
a = one-quarter the subtending angle of the arch ring.

It will be found that although the actual deflections at the different sections vary greatly from top to bottom of the dam, a coefficient of deflection, that is, the multiplier of the unit stress, is nearly constant for the different sections and for the usual subtending angles, probably quite as nearly constant as the elastic modulus would be.

Short-cut methods, therefore, can be used in special cases. For instance, the writer found that for a dam with a radius of extrados of 275 ft., the deflection, in inches, was about 3% of the unit stress, in tons, for arch action. This gave a simple and quick way of comparing the results of variations in the load curve and dam section. If the deflections are known, the unit stresses easily follow and *vice versa.*

Deflection caused by drop in temperature can be investigated in a similar manner, if the temperatures of the different parts of the dam can be determined The author's suggestions for more data on the actual deflections of arched dams might well be applied also for more data of temperature measurements in dams. By means of the simple methods outlined, but with a few trials, almost any of the properties of any assumed section for a dam can be determined with a sufficiently close approximation for practical purposes. The accuracy of the well known cylinder theory as applied to thin cylinders unrestrained at the base is easily demonstrated. When applied to thick masonry dams, restrained at the base and in competition with more efficient resisting forces, however, it has its limitations. Nevertheless, the simple cylinder theory for arch dams is too useful for preliminary work and approximate estimates to be scrapped at present. It can be made simple and attractive by short-cut methods for application to special cases, for approximate results.

For instance, to illustrate its great simplicity of application in special cases: If the radius of the extrados of the arch is 320 ft., the stress, in tons, by the cylinder theory, at any horizontal arch slice 1 ft. thick, at a depth of h ft. from the surface of the water, would be, stress $= 10 \times h$. If the radius is 160 ft., it would be, $5 \times h$. This illustrates also the importance of as small a radius for the arch, as is practicable.

* "Analysis of Stresses in Lake Cheesman Dam", *Transactions,* Am. Soc. C. E., Vol. LIII (1894), p. 108.

† "Arched Dams", *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

Even for approximate purposes, the cylinder theory must be used with judgment and with low unit stresses in compression. If the unit stresses vary from a little less than 10 tons per sq. ft. to a maximum of not more than 20 tons per sq. ft., in the different horizontal sections of the dam, the results will probably be safe and not extravagant. However, more accurate methods for important matters should also be used in the interests of economy and certainty of results, as the author has suggested.

It has been intimated somewhere that wedge action is the chief resistance in the bottom portions of the arch dam. A curved wedge, however, is essentially an arch. The faces of the wedge cannot slide wedge-like on the sides of the valley, but are cemented to the rock. It is as logical to assume, in pure horizontal arch action, that the bottom horizontal arch slices are free to slide on each other, as to assume that the slices higher up are free to slide. Practically, none of them actually does slide and cannot slide. Shearing strength is sufficient to prevent that. The fact is that arch action in the bottom portions of the dam is outclassed by more efficient resisting forces.

The writer is of the opinion that tension in the up-stream face of arched dams should be limited to a maximum of about 50 lb. per sq. in. Generally, the dam would be cemented into the rock bottom at the up-stream edge to a depth of several feet, which would give some shearing resistance and permit a small allowance for tension in the up-stream face. It should be noted that there is comparatively little opportunity for arch action, until there is a little tension in the up-stream face of the dam. The percentage of the load that would be carried by arch action depends on the conditions. Without tension, however, in a extrados of 275 ft. radius, considered by the writer, it would be about 70% at the top, dropping rapidly to about 10% at mid-section and to less than 1% near the bottom.

Arch action is most efficient at the top of a dam. Cantilever action is least efficient at the top. Simple vertical beam action, by carrying some bottom load to the upper horizontal arches as well as much more into the rock bottom, utilizes arch action where it is most efficient. Gravity action is the most efficient in all the middle sections and takes care of much the greater part of the load. Where the cross-section of the valley is of a rather flat V-shape, flattening out at the top, it will generally be economical to locate a portion of the ends of the dam on tangents to the main central arch. The radius of the arch should be made as small as practicable, the stress being directly proportional to the radius.

Shortening the radius of a dam of the constant radius type tends to bring the ends around parallel to the contours on the sides of the valley. Making a portion of the ends on tangents to the main central arch tends to cut the contours more nearly at right angles and shortens the dam. These tangents, of course, should be of full gravity section; but if the top of the arch dam has a substantial, or practically full, gravity section, it would work in nicely on the shallow ends.

The constant radius dam with tangent abutments is an approach toward the ingenious "constant angle dam"* of L. R. Jorgensen, M. Am. Soc. C. E.

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 685.

The "constant angle dam" will generally be more economical of material; but as nearly all the surfaces up stream and down stream are substantially warped surfaces, the construction difficulties are slightly greater and the appearance generally less pleasing. The "constant radius dam with tangent abutments" is between the "constant radius dam" and the "constant angle dam", and is likely to be the most satisfactory in many cases, all things considered.

Topographical conditions, of course, would determine the most suitable type of dam for any particular location. As the radius of the arch increases, due to topographical conditions, the reduced gravity section approaches the full gravity section, and when the radius is much more than 300 ft., or the length is more than would be subtended by an angle at the center of about 120°, a full gravity section would probably be advisable.

There is no question but that a straight gravity dam, with good design, good materials, and good workmanship, can be made entirely safe. If the arched plan costs materially more, due to increased length, or any other reason, it could not in general be justified. If, however, the arched form costs no more than the straight gravity dam, it would undoubtedly be justified, as giving an added insurance against failure, because if, for any reason, gravity action fails, arch action stands by, competent to do the work. Ordinarily, however, arch action does not get much opportunity to work until there is some tension in the up-stream face of the dam. This is particularly true for the bottom portions where arch action is surpassed and greatly relieved by more rigid resisting forces.

The writer trusts that the author's appeal for data on the deflections of dams will produce results, and that he will continue his good work in developing and simplifying mathematical expressions for the properties of arch dams. Measured deflections to be of value must be taken with great accuracy. The striking feature about the matter is the relatively minute deflections compared with the enormous forces involved.

L. Standish Hall,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has been impressed by the efforts made in the last few years to improve the theory used as a basis of the design of the arch dam. There is probably no engineering structure, considering the magnitude, in which precedent plays such an important part in the design. The formula that is most generally used in proportioning an arch dam, is the so-called cylinder formula. A brief comparison of the conditions under which this formula is applicable with those existing in an arch dam, will show that it is materially defective.

However, many dams have been designed by the cylinder formula, and the principal arguments to support its use are the ease with which it may be applied and the fact that no arch dams designed by it have failed. The fact that these dams have been designed according to a defective formula does not necessarily infer that they are unsafe. A large "factor of safety" is usually allowed so that the unit stresses in the concrete are much below those used in building construction where the distribution of the stresses can be calculated more accurately.

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A few years ago, the writer made a study of the arch dam and arrived at conclusions, quite similar as regards the requirements of formulas to cover the actual static conditions as those developed by Mr. Noetzli. It is believed that the author's method is sound from a theoretical standpoint, although with such a structure as an arch dam, in which static conditions are so completely indeterminate, it would appear to be impossible to develop from theoretical considerations alone, formulas that will apply accurately to actual conditions.

Real progress can only be made in the method of designing arch dams by securing more deflection measurements and other tests on existing structures, so that sufficient data may be collected to enable engineers to modify the theoretical formulas (in case such modification is found to be necessary) by certain empirical coefficients derived as the results of tests. The collection of such data could probably be best undertaken by some department of the State or Federal Government. Such observations would be assured of greater standardization in method and a greater uniformity of results.

The writer has not seen typical cross-sections of dams designed according to the author's principles. It is believed that such a presentation is essential to a clear understanding of the variations between his methods and the results obtained by the cylinder formula. It is understood that the cross-section of an arch dam will vary with the height and radius, but if the cross-section of some existing dam were shown, together with a design by the author for the same location, an interesting comparison could be made. The Barren Jack Dam or the Salmon Creek Dam which were shown, might be used in making such a comparison, and the writer hopes that the author will include some such presentation in his closing discussion.

William A. Miller,* Assoc. M. Am. Soc. C. E. (by letter).—The writer has been much interested in Mr. Noetzli's further contribution to the literature on arch dams. Throughout, it is clear that one aim of the author is to place before the Profession a solution of the arch dam problem, as accurate as possible, without involving laborious algebraic or graphical operations. Although the writer has not especially considered the arch dam, he has studied the arch rib as a structural element from this same aspect, and he will contribute to the discussion of this paper by quoting the results of those portions of his investigations that are relative to the application of the arch rib as met with in the arch dam.

If the same assumptions are made in the derivation of Equation (17) as in arriving at Equation (10), then Equation (17) should read:

$$H_t = \frac{c\,T\,l\,E}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{A} + 3\int \sin^2 \alpha \frac{d\,L}{A}} \qquad (23)$$

for the displacement of the point of application of H_t (that is, Point O' in Fig. 7 *b*, of Mr. Noetzli's paper, "Gravity and Arch Action in Curved Dams")†

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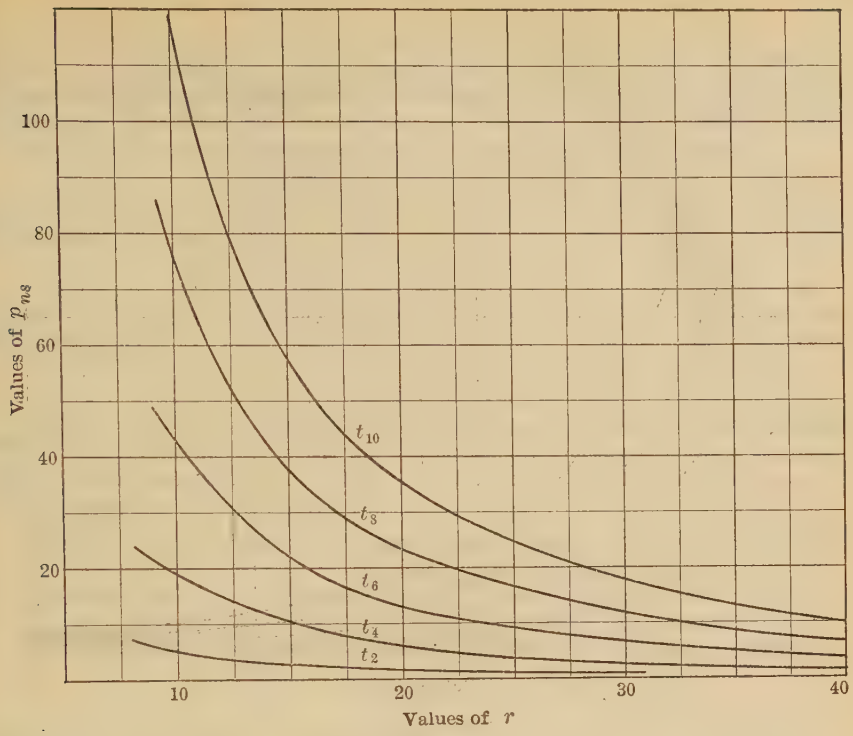
† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 27.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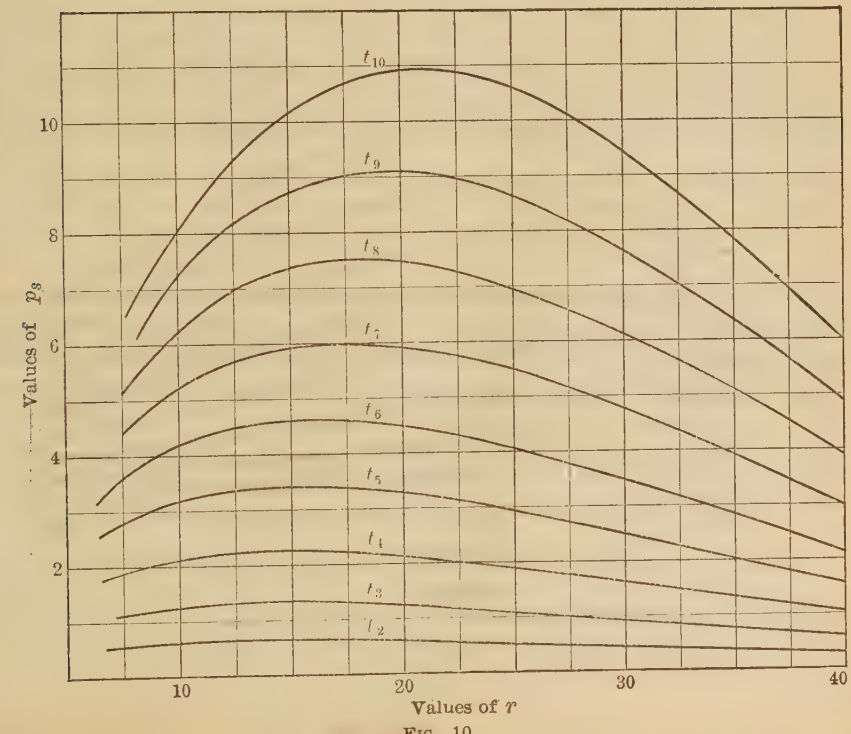
80
60
40
20
Values of p_{ns}
t_{10}
t_8
t_6
t_4
t_2
10
20
30
40
Values of r

FIG. 9.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8
6
4
2
Values of p_s
t_{10}
t_9
t_8
t_7
t_6
t_5
t_4
t_3
t_2
10
20
30
40
Values of r

FIG. 10.

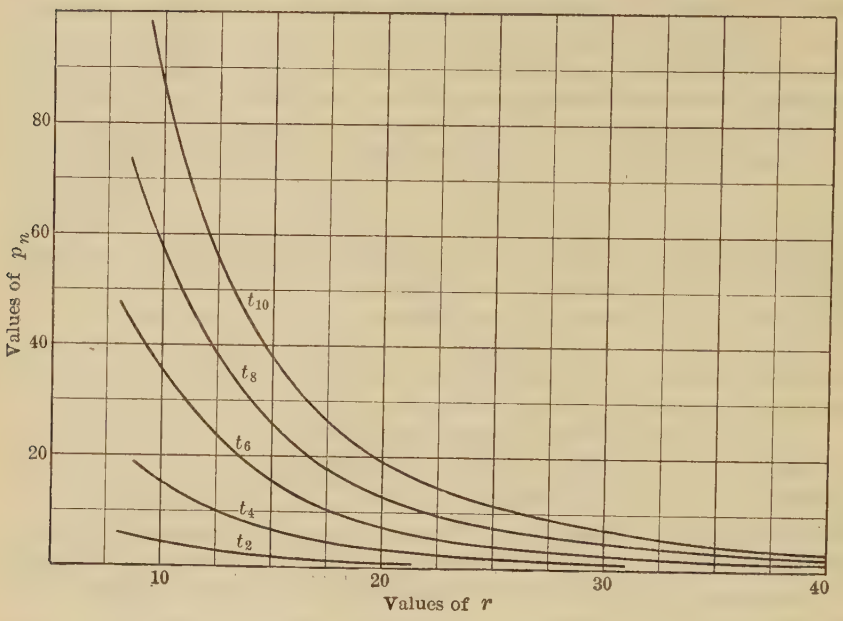
80
60
40
20
Values of p_n
t_{10}
t_8
t_6
t_4
t_2
10
20
30
40
Values of r

FIG. 11.

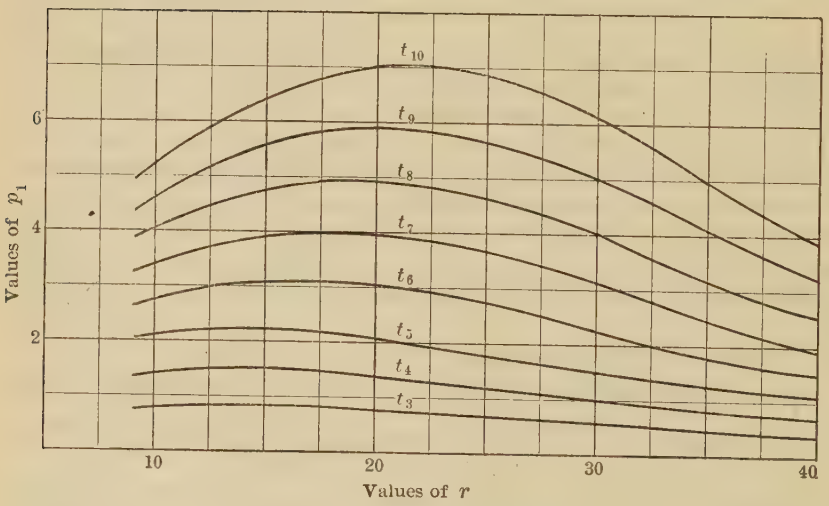
6
4
2
Values of p_1
t_{10}
t_9
t_8
t_7
t_6
t_5
t_4
t_3
10
20
30
40
Values of r

FIG. 12.

due to temperature rise $= c\,T\,l$; and, due to H_t acting at O', the bending moment at $(x\,y) = M_x = -H\,y$, the axial force at $(x\,y) = N_x = H\cos\alpha$, and the shearing force at $(x\,y) = S_x = -H\sin\alpha$, giving the displacement of O' due to M_x as,

$$\int M_x\,y\,\frac{d\,L}{E\,I} = -\int H_t\,y^2\,\frac{d\,L}{E\,I},$$

due to N_x as,

$$\int N_x\cos\alpha\,\frac{d\,L}{E\,A} = -\int H_t\cos^2\alpha\,\frac{d\,L}{E\,A},$$

and due to S_x as,

$$\frac{6}{5}\int S_x\sin\alpha\,\frac{d\,L}{G\,A} = -3\int H_t\sin^2\alpha\,\frac{d\,L}{E\,A},$$

where G is the rigidity modulus of the material, and taken here for concrete $= 0.4\,E$.

As the displacement of O' is zero,

$$c\,T\,l - \int H_t\,y^2\,\frac{d\,L}{E\,I} - \int H_t\cos^2\alpha\,\frac{d\,L}{E\,A} - 3\int H_t\sin^2\alpha\,\frac{d\,L}{E\,A} = 0$$

or,

$$H_t = \frac{c\,T\,l}{\int y^2\,\frac{d\,L}{E\,I} + \int\cos^2\alpha\,\frac{d\,L}{E\,A} + 3\int\sin^2\alpha\,\frac{d\,L}{E\,A}} \quad \ldots\ldots\ldots (24)$$

which for the constant, E, would take the form shown in Equation (23). It is to be noted that the denominator of Equation (24) is the displacement of O' due to unit load acting at O' and in the line of action of H_t; it is independent of the loading on the arch. The numerator is the displacement of O' due to the applied loading or, as in this particular case, temperature change, that is, if R is the displacement of O', due to any loading whatever, then the thrust is,

$$H = \frac{R}{\int y^2\,\frac{d\,L}{E\,I} + \int\cos^2\alpha\,\frac{d\,L}{E\,A} + 3\int\sin^2\alpha\,\frac{d\,L}{E\,A}} \quad \ldots\ldots\ldots (25)$$

It will be clear that the author's coefficients, k_t, are slightly in error. Fig. 12 shows the percentage error involved. The writer presents the foregoing, not with the idea of emphasizing small matters, but rather to make clear what follows. Considering the writer's Equation (25) to give the accurate value of the thrust, then,

$$H = \frac{R}{\int y^2\,\frac{d\,L}{E\,I}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

when deformations due to axial and shearing forces are neglected;

$$H = \frac{R}{\int y^2\,\frac{d\,L}{E\,I} + \int\cos^2\alpha\,\frac{d\,L}{E\,A}} \quad \ldots\ldots\ldots\ldots (27)$$

when only shearing force deformations are neglected;

$$H = \frac{R}{\int y^2 \frac{d\,L}{E\,I} + 3\int \sin^2 \alpha \frac{d\,L}{E\,A}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

neglecting only axial force deformations; and using the approximation in Mr. Noetzli's Equation (17),

$$H = \frac{R}{\int y^2 \frac{d\,L}{E\,I} + \int \frac{d\,L}{A}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

For the rib as found in the arch dam the form in Equation (29) is equivalent to using Equation (25), but substituting unity for the coefficient, 3, of the third term of the denominator.

Let l = span;

r = the ratio, $\frac{\text{rise}}{\text{span}}$, expressed as a percentage;

p_{ns} = the percentage over-estimate of the value of H using Equation (26);

p_s = the percentage over-estimate of the value of H using Equation (27);

p_n = the percentage over-estimate of the value of H using Equation (28);

p_1 = the percentage over-estimate of the value of H using Equation (29).

Let t_1 denote a rib having a thickness equal to 1% of the span, and let t_2 denote a rib having a thickness equal to 2% of the span, and, similarly, for other rib thickness; then Figs. 9, 10, 11, and 12 show values of p_{ns}, p_s, p_n, and p_1, respectively, for a series of ribs having values of r ranging from 10 to 40 and rib thicknesses varying from 2 to 10% of the span; the axes follow closely the axes used by Mr. Noetzli.

The diagrams call for little or no comment as they explain themselves. From Fig. 9, it is clear that flexural deformations alone should be considered, only for thin, high, rise ribs. Fig. 11 shows that the neglect of axial force deformations has little to commend it. Fig. 10 shows that including flexural and axial force deformations and neglecting those due to shearing forces gives percentage error in the value of thrust, almost constant for variation of rise, but increasing with increase of rib thickness. A study of Fig. 12, however, shows that a greater degree of accuracy is obtained by the use of Equation (29), which is the same as Mr. Noetzli's Equation (17). Not only does this expression give the closest approximation to the true value of the thrust, but the calculations are much simplified—a most desirable combination from the designer's viewpoint—for $\int \frac{d\,L}{A}$ replaces $\int \cos^2 \alpha \frac{d\,L}{A}$.

Chauncy Wernecke,* Assoc. M. Am. Soc. C. E. (by letter).—The author's appeal to the Profession to assist in obtaining all possible data regarding de-

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flections, temperature variation, and conditions of existing single arch dams should receive active support. Sufficient measurements on arch dams should go a long way to establish a more accurate method of design. Having such data, effective guidance could be given to further research with models. The degree to which the principle of similitude would apply, and the limiting size of models for reliable results, could be determined. Having thus established the reliability of models, the design of important structures might well be made from them, and the influence of the shape of the canyon could be determined in a most complete manner. The by-product value of such work would be considerable in any particular case. Such a method of design might seem costly at first, but in view of the possible saving in important structures, the extra expense would be well invested. Handling the problem in this manner would go far toward establishing faith in the economical arch dam.

There are two factors extraneous to scientific considerations in the design of an arch dam: First, a psychological factor resulting from lack of knowledge, and, second, a political factor injected when there are active opponents of the project in the community. The first consideration is partly basic to the second. Engineers lack knowledge as well as others. It is not to be expected that the knowledge and faith in the arch, as applied to bridges and buildings, will quickly transfer to a different use. It is to be expected that, even for an engineer or architect whose knowledge of the arch is extensive and whose faith in it is great, it will take some time for the same state of mind to form about the arch dam. For the average citizen, a proper faith in an arch dam would have to be carefully formed by well designed and honest propaganda. Otherwise, the second factor might cause considerable damage or delay to a project entertaining the use of such a structure. Such was the experience in 1904, with the Six-Mill Creek Arch Dam* for the water-works of Ithaca, N. Y., wherein the design afforded sufficient reason for a political filibuster to obtain public ownership of the water supply, and to reduce the dam from 90 ft. in height to 30 ft. The proportions of the Six-Mill Creek Dam were approved by a board of consulting engineers, but this had little effect.

It would seem advisable to initiate a co-ordinated research which might well begin with a thorough and impartial examination of arch dams. It is gratifying to note the interest in the subject, as measured by the space given it in the publications of the Society and other scientific journals.

W. H. R. Nimmo,† Assoc. M. Am. Soc. C. E. (by letter).—The papers recently presented by Mr. Noetzli have brought out many interesting points bearing on the design of arch dams, and the solution of the circular arch under water loads by William Cain, M. Am. Soc. C. E.,‡ is also a valuable contribution on the subject.

The writer recently had occasion to attempt the elastic analysis of the stresses in a proposed high arch dam, but arrived at the conclusion that the inaccuracies in the assumptions, which were necessary, rendered any numerical results valueless for the purpose of practical designing. For the same reasons,

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 183.

† Asst. Civ. Engr., Hydro-Elec. Dept., Hobart, Tasmania.

‡ See p. 233.

the writer is unable to accept the startling results arrived at by the author and, in the absence of definite evidence from observations, is not convinced that most arch dams are cracked.

Most investigators have adopted the method of equating the deflections of elementary horizontal arches and vertical cantilevers in order to find the proportions of water load carried by the two systems, the stresses in each system being then investigated separately. The method neglects other possible systems whereby part of the water load may be carried, but this probably causes an error on the side of safety and valuable results might be obtained if no other errors crept in. Unfortunately, the assumptions usually made, neglect, or do not accurately allow for, the following considerations:

(1).—Temperature variations, which are both uncertain in amount and irregular in distribution, the temperature in the body of the dam lagging behind the atmospheric temperature and this time lag is not the same at all levels in the dam.

(2).—Shrinkage due to setting of concrete.

(3).—Swelling due to soaking of concrete at the up-stream face.

(4).—Yielding of foundation due to eccentric vertical loading.

(5).—Yielding of foundation horizontally.

(6).—Yielding of abutments.

(7).—Deformation due to shear in cantilever.

(8).—Ordinary theory of flexure not strictly applicable to a triangular cantilever.

(9).—Cantilever assumed to be contained between parallel instead of radial planes. In a constant angle arch dam, the thickness at the lower levels becomes a large proportion of the radius, and this assumption involves considerable error in the value of the moment of inertia of the cantilever.

(10).—Formulas generally used for deflection of arch are not applicable when the arch thickness becomes appreciable in comparison with the radius.

(11).—Modulus of elasticity not the same for tension and compression.

(12).—Modulus of elasticity is not constant, but increases with the stress.

(13).—Effect of time factor in permitting increased deformation in the concrete.

(14).—Effect of lateral deformation.

(15).—Wedge action near base.

(16).—Possible arching in inclined planes.

Many of these points have been brought out in the previous discussion.

The effect of Assumptions 3, 4, 5, 7, 8, 9, 12, 13, and 14 would be to increase the deflection of the cantilever, and the combined effect may be sufficient to account for the differences in deflections as calculated by the author's method and as observed in existing arch dams.

By further complicating the calculations, it would be possible to develop a theory including the effects of Assumptions 7, 8, 9, 10, and 14, and the effects of yielding of foundation and abutments might be allowed for approximately. The effect of wedge action near the base and inclined arching, the writer believes, would be to relieve the horizontal arches and cantilevers, and a theory which neglects these, might still be of practical value.

The effects of Assumptions 1, 2, 3, 11, 12, and 13 are of prime importance and cannot be neglected, but, unfortunately, engineers are lamentably ignorant regarding them. Even if they could be accurately taken into account and the proportions of the load carried by the arches and cantilevers at the crown, at a given temperature, accurately determined, the difficulty would still remain, that these proportions would not hold for any other part of the dam, even at the same temperature. In reality, the load on the arches is not a uniformly distributed load, but an irregular one, which also varies with the temperature.

If an entirely satisfactory method of obtaining the stresses in an arch dam was available, it would be possible to design arch dams as reinforced concrete structures, which possibly would be more economical than dams of plain concrete. There is great need for more extensive observations on existing arch dams, including measurements of temperature, deflections, and strains, supplemented by further laboratory tests on concrete mixtures in regard to their elastic properties. When sufficient data of this kind are available, reliable formulas for the more economical design of arch dams may be developed. In the present state of the knowledge, it is probable that responsible engineers will continue to design arch dams on the assumption that the arches carry all the load, and it is for the purpose of more nearly realizing this condition that the system of grouting joints under pressure has been introduced.

The principal objection to the pressure grouting of joints arises from a belief that uneven filling of the joints will cause eccentricity of the arch thrust, which will result in dangerous bending moments. Although it is not probable that the grouting of joints can be made 100% efficient, it should be possible, with sufficiently close spacing of the grout pipes, to insure that such air pockets as might remain, would be small and well distributed over the cross-section, in which case, no appreciable eccentricity of the thrust would be caused. A small increase in the intensity of the arch thrust at the joint might result, but this would not be serious, and it is likely to arise, even if the joints are not grouted, from débris falling into the joints while they are open. In a dam, the joints of which open and close periodically, the effect of varying temperatures in the lower arches, combined with the resistance of the cantilevers, will cause the upper arches to close on slightly varying alignments, unless they are accurately keyed at the joints. In other words, the keys will be carrying shear stresses before the joints in the upper part of the dam have closed, which means that the surfaces of the keys may be sliding against each other under pressure, and this is certainly not a desirable condition. If satisfactorily carried out, there can be little doubt about the advantages of grouting joints under pressure.

In dealing with the deflections of the Salmon Creek Dam, the author finds, by subtracting the deflection at Elevation 1095 from the mean of the deflections at Elevations 1075 and 1115, that the cantilever has suffered a deflection of 0.18 in. in a length of 40 ft. The method of measuring the deflections has not been described in detail, but if they were made by angular measurements with an ordinary transit instrument, reading to, say, 10″ of arc, they may easily be in error by ± 0.05 in., in which case the 0.18 in. previously mentioned, may be greatly in error.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E., AND ALBERT E. GREENE, ESQ.† (by letter).—At the time Mr. Noetzli's interesting paper was published, the writers were engaged in designing an arch dam about 85 ft. high and of 325 ft. radius. The available literature on the subject had been studied, various designs of thin arch dams had been examined, and the conclusion had been reached that the usual method of designing by the formula, $T = P R$, was not even approximately correct and that all existing thin arch dams of considerable span that are not anchored to the rock by reinforcing, must have opened a horizontal joint at the base. The deflection measurements of the Barren Jack and the Salmon Creek Dams seem to prove the failure at the base, beyond question. As it was not desirable to attempt to fix the base of the proposed dam, it was decided to make a hinged joint there and to allow the dam to rock freely with the imposition of the water load and, particularly, under changes of temperature. The site is in a gorge with a level bottom, and the arch dam is sprung between vertical abutments at the side-walls. For this particular condition, the stresses in a dam of any chosen cross-section can be computed readily.

The dam is assumed to consist of a system of vertical needles supported by a system of horizontal arch ribs, as shown in Fig 13, the assumption made by every one who has attempted a solution of the problem, as far as the writers are aware. Because of the long radius and the consequent thickness of the dam, the needles of the design in question are much more rigid than the arch ribs and remain practically straight, transmitting excesses of pressure from one arch to another when the arches deflect under load. The deflection of any rib, therefore, is proportional to its distance above the hinge at the base and, as the unit stress in any rib is proportional to the deflection of that rib, the unit stress at any point of the cross-section is proportional to the distance of that point above the hinge; that is, the thrust in the arch dam is distributed over the section in the same manner as the stress is distributed, under positive bending moment, over that part of the section of a beam lying above the neutral axis.

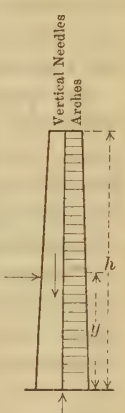

FIG. 13.

To find the magnitude of the rib stresses, consider the equilibrium of one of the needles lying between two radial planes, 1 ft. apart, at the up-stream face of the dam where the radius is R. The needle is in equilibrium under the action of the water pressure, its own weight, the reaction of the arch ribs, and the reaction at the base. The two sides of the needle are cross-sections of the dam and are inclined to each other at an angle of $\frac{1}{R}$, and it is on these sides that the arch thrusts are actually applied, as shown in Fig. 14, and not on the face, as shown in Fig. 13. Let f be the normal unit stress on the cross-section at an elevation, y, above the base, and let f_c be its value at the crest. The component of f on the median plane of the needle, therefore, is

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† Ann Arbor, Mich.

$\frac{f}{R}$, and on any elementary area of the cross-section dA distant y above the base, there is a force of $\left(\frac{f}{R}\right) dA$ opposing the water pressure. Taking moments about the lower end of the needle or the base of the dam and equating the overturning moment of the external forces to the resisting moment of the components of the rib stresses in a manner similar to that followed in deriving the expression for the moment of resistance of a beam, since $f : y = f_c : h$, we have,

$$M = \int_0^h \frac{f}{R} y \, dA = \frac{f_c}{R h} \int_0^h y^2 \, dA = \frac{f_c I}{R h}$$

or, the arch stress at the crest is,

$$f_c = \frac{M h R}{I}$$

in which I is the moment of inertia of the vertical cross-section of the dam about an horizontal axis at the base, and M is the moment of the external forces, the water pressure, and the weight of the structure, acting on a vertical slice of the dam 1 ft. wide at the up-stream face.

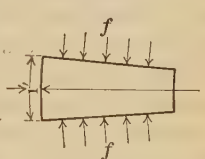

FIG. 14.

It appears that the most economical section for a dam of this type is one with a large value of I and that, instead of making the crest narrow and increasing the thickness toward the base where the arch stress is zero, good designing will place as much material as possible near the crest. A wall of uniform thickness, therefore, is preferable to the usual trapezoidal section for dams of moderate height in a box canyon.

Attention must be given to the design of the needles which, if of the usual form, will require reinforcing on the down-stream face to prevent breaking. The line of thrust can be kept within the cross-section and the necessity of reinforcing obviated by curving the structure in the vertical plane, as one of the writers did in the Six-Mile Creek Dam.* Where reinforcement has been used in the past, the arches have been reinforced and not the needles which particularly need it, if the dam is to act as a monolith.

Temperature stresses in dams of this type were found to be of much importance, and they are undoubtedly so in all arch dams. In the design in question, it was computed that a change in the temperature of the concrete of 30° Fahr. would deflect the crest about twice as much as the water pressure alone, so that, assuming the dam to be built with expansion joints, which are to be filled in cold weather, the dam might be expected to deflect up stream in the summer with full pond, as far as it would deflect down stream in winter, both deflections being measured from the unstressed position.

The formulas developed for this dam placed between two vertical abutments should not be applied to thin arch dams in a valley with sloping sides. Writers who have attempted an analysis of the general problem have assumed

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 183 *et seq.*

that every arch rib carried a uniform radial load. As the water pressure on any rib may be carried in part by that rib and may be distributed in part by the needles to other ribs, there is no good reason for thinking that a given rib is uniformly loaded, except in the case where all the needles are of the same length.

An attempt was made to analyze the two dams the deflections of which are discussed by the author, on this basis, and it appeared that the needles were not strong enough to distribute the water loads between the various ribs. It is believed that the upper arches of a dam in a V-shaped valley are relieved of part of the load which would be indicated by their deflections if the assumption previously mentioned was correct, because they do not take the form the computer thinks they should. For this reason, the writers do not wholly agree with the author in his conclusions about computing the stresses from the deflections. They, however, would congratulate him on this and his previous paper,* which are valuable additions to the progress being made in the analysis of arch dams.

While on the subject of arch dams, the writers would place on record some unusual examples which do not appear in any of the published lists they have seen. These dams are timber structures now or formerly existing on Taghanic and Six-Mile Creeks, near Ithaca, N. Y., where the geological formation consists of shale in horizontal layers with vertical fracture joints, which affords a channel with a level bottom and nearly vertical sides.

The first of these structures is the Van Natta Mill Dam on the Six-Mile Creek, at Ithaca, N. Y., which property was acquired by the Ithaca Water-Works Company in 1892. Previous to that, it had been used as a grist-mill and the dam had been in place for many years. The date of its construction is unknown to the writers. It continued in service until the great flood of June, 1905, when, with a head between 7 and 10 ft. above its crest, it was carried away.

The dam which rested on rock, consisted of a spillway between two masonry abutments. The spillway was 2.35 ft. thick, built of planks laid flat and spiked together and was anchored to the rock by iron rods about 10 ft. apart, extending several feet into the body of the work. The length of the crest of the spillway, on center line, was 90.25 ft., with a down-stream chord length of 86.4 ft. The north abutment was from 4 to 6 ft. thick, whereas that at the south was about 2.5 ft. thick and extended in a curve about 16 ft. to a vertical rock face against which it abutted, its height varying from 3 to 7 ft., of which 2 ft. was above the crest of the spillway. The up-stream radius was 100.66 ft. The crest of the spillway was inclined, the down-stream edge being 9 in. higher than the up-stream corner. For 56 ft. of its length, the spillway was from 17 to 18 ft. in height, and in the 16 ft. next the north abutment, the rock was inclined so as to give it a height of 12 ft. at that abutment. On the south, the rock inclined through the last 18 ft., giving a height of spillway at the abutment of only 5 ft. At the time of failure, the flow in the creek was more than 8 000 cu. ft. per sec., and the structure had withstood previous floods of more than half that quantity.

* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

The second dam was built by the Ithaca Water Company in Six-Mile Creek about 600 ft. down stream from the so-called Six-Mile Creek Dam, and varied in height from 5.6 to 6.5 ft., had an up-stream radius of 109 ft. and a span of 94 ft., with an uniform thickness of 1.42 ft. It was built of two walls of 2 by 4-in. timbers, laid flat, with the intervening 9-in. space filled with concrete. It was anchored to the rock with rods about every 5 ft. This dam, which was in the gorge,* was in service four or five years and passed floods of 2 to 4 ft. over its crest. During the construction of the Six-Mile Creek Dam, about one-third of it was removed to drain the gorge and was later replaced. It was tipped over at the center and partly carried away in the flood of 1905, which passed over the dam above, with a head of about 10 ft. It is not known at what time during the flood this dam failed. The discharge of the creek during the flood was at the rate of more than 8 000 cu. ft. per sec., and this dam had withstood a flood of about 4 000 cu. ft. per sec., previously.

In 1904, one of the writers built the third dam which is across Taghanic Creek above Taughannock Falls and may be seen from the Lehigh Valley Railroad. The up-stream radius is 120.5 ft., the span 150 ft., the height is approximately 6 ft. throughout its length, and the thickness is 1 ft. It is built of oak plank laid flat and spiked together, and anchored to the rock by 1-in. bolts every 4 ft. Each alternate bolt goes to the crest, the others going through the sill only. They are set 1 ft. deep into the rock.

During a heavy flood in March, 1912, ice was brought down the creek in blocks 2 ft. thick and more, and lodged against the dam, and the head-water rose to a height of 5 or 6 ft. above the crest. The accumulated ice and water folded the dam over at its center, until the upper part of the section lay almost at right angles with the lower part. When the flood subsided, the deformed part was jacked back into position, a timber brace was set against its center and the structure continued in service.

Another structure which may be of interest in this connection, is Campbell's Dam in Cascadilla Creek at Ithaca, designed by one of the writers and built by the owners in 1904. It is of concrete, reinforced with miscellaneous iron and steel rods. The up-stream radius is approximately 70 ft., the crest is 29 in., and the base 4 ft. thick. The up-stream face inclines down stream 1 in. per ft. The length of the crest is 51.6 ft. at the down-stream edge. The abutments which are part of the arch and about 3 ft. above the crest, are 25 ft., and 11.3 ft. long at the crest level. For 35 ft. of its length, the dam is 25 ft. in height and decreases to 17 ft. at the south abutment, and runs to zero at the ends. This dam has had a flood of about 5-ft. head over it.

The Van Natta, Taughannock, and Campbell Dams were all near the crest of a fall, so that the back-water could not rise more than 2 or 3 ft. below, in any case. The dam of the Ithaca Water Company was undoubtedly submerged in the big flood.

F. A. Noetzli,† Assoc. M. Am. Soc. C. E. (by letter).—By a fortunate coincidence, the writer's investigations on arch deflections and stresses were

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), Plates IX and X, pp. 185, 187.

† Chf. Engr., Beckman and Linden Eng. Corporation, San Francisco, Calif.

published at the same time as a paper* on a similar subject by William Cain, M. Am. Soc. C. E. The writer concentrated his efforts on the development of simple methods and approximate formulas that might be more easily understood and used by hydraulic engineers engaged in dam design, whereas Professor Cain has effected, in a brilliant manner, the theoretically correct solution of the determination of the stresses in the arching parts of arch dams. A comparison between the two methods discloses the fact that in many cases results obtained by the writer's approximate formulas are almost identical with those obtained by the exact method of Professor Cain. In other cases, some discrepancy exists, as was naturally to be expected. However, the difference for true arches, within practical limits, is seldom great enough, that it could not be considered practically negligible when compared with the uncertainties involved in the assumptions as to the modulus of elasticity, degree of shrinkage of concrete, range of temperature, etc. The greatest discrepancy is found where the two methods are applied to thick and flat arches such as generally occur in the lowest parts of arch dams. It is questionable, however, whether any known arch formula can be applied to very thick arches without considerable modification. Professor Cain has shown, for instance, in an arch 40 ft. thick, of 135 ft. radius, and with a central angle of 40°, that only about 2½% of the radial load is supported by what is commonly understood as arch action. On page 249 of his discussion of Professor Cain's paper referred to previously, Mr. Mensch has indicated why such an "arch" cannot act as a horizontal beam, and the writer, on page 262 of his discussion of the same paper, has tried to illustrate the reasons why "secondary arch action" within the arch may interfere with true arching.

In any arch for which it is possible to inscribe, for instance, another arch of half the thickness and having a radius of less than half the radius of the primary arch, so-called secondary arch action is likely to occur. The reason for this lies in the fact that the stresses in the secondary arch are theoretically less than those in the primary arch for the same load, and Nature makes use of this fact by what is known as the "principle of least work". Not only one secondary arch is formed within the body of the primary arch, but a series of elementary arches, whereby the load can be supported with a minimum of elastic deformations according to the previously mentioned "principle of least work."

Furthermore, an investigation made by the method of combined cantilever and arch action will generally show that, in the lower parts of dams, arch action is very small, most of the load near the foundation being supported by the vertical cantilevers. These considerations unquestionably argue for caution in applying any kind of arch formula to the lowest parts of arch dams, and if such extreme cases are excluded, it will be found that the writer's approximate formulas given in his paper may well be used to illustrate the relation between deflections and stresses in arch dams.

In order to avoid misunderstandings, it may be desirable to fix some limits outside of which the approximate formulas cannot be expected to furnish other than rough approximations. In the writer's judgment, the deflection stress

* "The Circular Arch under Normal Loads", p. 233.

Formulas (1) to (9) should not be applied without modification to such arches for which the proportion between thickness and rise of arch is nearly unity or even larger, that is, for which, say, $\frac{t}{h} \geq 0.8$.

Mr. Mensch raises the interesting question of why an arch dam, in which according to the writer's theory the cantilever appears to have developed cracks, does not leak. Leakage is prevented by the weight of the masonry. Although a dam may have failed in gravity action and developed horizontal cracks at the up-stream face, the concrete near the down-stream side is under high compression. Therefore, cracks, due to bending of the cantilever, will hardly ever extend through the dam for this same reason. The conditions are different for circular reinforced tanks for which the vertical walls have not been tied adequately to the base to take care of the vertical bending moment. In such faultily designed tanks, the bending moment at the base of the thin and light vertical walls generally starts a crack, and uplift and shear do the rest, thus effecting a measurable movement between vertical walls and base, and leakage is almost certain to occur. In dams which may be from 30 to 40 ft. thick at the base and which are prevented from considerable movements by the relatively close vicinity of the side abutments, there is little chance for a crack to extend all the way through, as the weight of the masonry above will always tend to close a crack and seal tightly the down-stream part.

Mr. Mensch further asks why an arch, which according to the writer's approximate method ought to show serious defects, does not fail? It may be stated that the example to which Mr. Mensch makes particular reference is a rather extreme case,* that is, an "arch" for which the thickness is about 1.3 times the rise $\left(\frac{t}{h} = 1.29\right)$. Furthermore, it is to be considered that any formula, if applied to a hingeless, unreinforced concrete arch, gives reliable results only if the arch has no open cracks. If calculations show that excessive tension occurs in such an arch at certain places, so that cracks may be expected on the tension side, such cracks act statically as a kind of "hinge" and the basic assumptions for a hingeless arch are void and, therefore, also the formulas based on such assumptions. If a crack (partial hinge) which, for instance, may have opened due to a rise of the arch temperature when the reservoir was empty, is closed entirely by the pressure of the rising water, the arches may again be considered as hingeless for purposes of stress calculations, although, of course, the crack still exists.

To make this point clear, assume an arch dam with reservoir empty, and such a decrease of temperature that the construction joints have opened. Either Professor Cain's exact formula for temperature stresses, or the writer's approximate formula, or any other formula, would furnish, for such a decrease of the arch temperature, certain stresses at the abutments as well as at the crown of the dam. As the construction joints are open, it is evident that no

* Such conditions were referred to by the writer in "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 134, to be examined by the exact theory of the elastic arch.

stresses can be transmitted through such open cracks, and both the crown and abutments of the arches may be without temperature stresses. However, due to the shortening of the arch sections resulting from the assumed drop of temperature, the two surfaces of each joint are turned in such a manner that they are no longer parallel. If the water rises and presses against the arches, the temperature being the same, the open joints will begin to bear unevenly against each other. For instance, at the crown, the parts farther up-stream may start to transmit the arch thrust from water pressure, while the joint may still be slightly open on the down-stream side. Eventually, the water pressure may be sufficient to produce compression over all the sections of the arch, and therewith the conditions for theoretical hingeless arch action are re-established, and the formulas apply for this condition although the arch is cracked.

Furthermore, it may happen that, in a solid unreinforced arch, the thrust has moved outside the middle third of the section and sometimes, theoretically, even outside the section itself. As soon as the tension produced by the eccentricity of the arch thrust is large enough to open a crack on the tension side, an artificial "hinge" is created and the arch thrust is automatically moved back and passes through the center of the "hinge". The assumptions as to hingeless arch, therefore, are changed for these conditions and the results from the formulas have to be interpreted accordingly.

Such considerations may clear some misunderstandings that certain members seem to have had from applying the formulas and conclusions of the writer with respect to the present conditions of certain arch dams. For arches that are adequately reinforced, the formulas apply within their limits of accuracy, without modification.

Professor Cain's numerical examples, by using his exact method and comparing the results with those obtained by the writer's approximate formulas, should prove instructive. Thus, the similarity of the results of respective calculations applied to the arch slice at Elevation 1115 of the Salmon Creek Dam is rather striking. The stresses at the crown were found to be:

	Cain.	Noetzli.
At the extrados, $f_c =$	468	461 lb. per sq. in. (compression)
At the intrados, $f_c =$	126	94 lb. per sq. in. (compression)

and at the abutments,

At the intrados, $f_c =$	656	645 lb. per sq. in. (compression)
At the extrados, $f_c =$	— 22	— 89 lb. per sq. in. (tension)

The difference between the two results appears to be slight if compared with the results obtained by the cylinder formula which furnishes, both at the crown and at the abutments, a uniform axial stress of 307 lb. per sq. in. (compression).

The point that the writer desired to bring out by the determination of the stresses from measured deflections was particularly to show the wide discrepancy between cylinder theory and actual conditions. It is immaterial, therefore, whether the results by the approximate formulas differ a few pounds per square inch from the theoretically correct values, as long as the apparent defects of existing arch dams may be brought, in an easily comprehensible manner, to the attention of engineers. In order to obtain more nearly correct

results by the writer's method, it would be necessary to determine the actual load on the arch by the method of combined cantilever and arch action. For illustrative purposes, however, the figures given may be considered as sufficiently accurate to arouse the interest of engineers in arch deflection stresses.

The writer is unable to agree with Professor Cain's interpretation of the part played by temperature deformations, if the stresses in an arch ring are to be determined from the deflections. The formula for the deflection, D, on page 316, seems to be incorrect as far as the part referring to temperature is concerned. This is evident from Fig. 18 (a) and Fig. 18 (b)* from which this formula was derived. It also differs considerably in its results from Professor Cain's exact formula for temperature deflections.

Furthermore, the writer questions whether the assumption of "dam simply supported at the base, with no bending moment there", will ever be valid in the case of high dams. Such a dam may have developed cracks and, therefore, a partial "hinge" near the center of the canyon, but such cracks hardly ever will extend all along the side abutments. Therefore, bending moments at the base are certain to exist in the vertical cantilevers nearer to the side-hills. Furthermore, the weight of the masonry of high dams will always exert a considerable moment at the base, which acts in an opposite direction to the moment produced by water pressure. The deflections of an arch are influenced materially, also, by the moment exerted by the weight of the dam and the neglect of this weight moment, made by the assumption of "no bending moment at the base", may lead to results more or less in error.

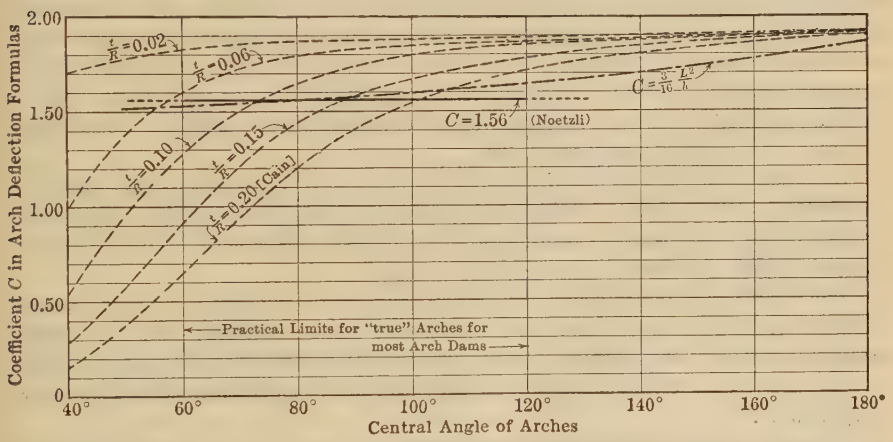

FIG. 15.

The theoretically exact deflection Formula (15) given on page 241 of the paper by Professor Cain, previously referred to, for which coefficients were given in his Table 1 (page 234), will be of great help in many cases. As a matter of comparison, those coefficients were plotted in Fig. 15, together with the coefficients, 1.56, and $\frac{3L^2}{16h}$, of the writer's approximate deflection formulas.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 80.

It will be noticed that the difference is comparatively small for most arches of the upper parts of arch dams where free arching may occur. In cases where accuracy is of great importance, Professor Cain's exact coefficients should be used for calculating deflections, in preference to the writer's approximate formula. In the writer's opinion, both formulas should be applied with caution for very thick or flat arches.

The writer fully appreciates the excellent results arrived at by Professor Cain in his admirable paper on "The Circular Arch under Normal Loads". Engineers who desire to design arch dams with the utmost degree of accuracy, will find satisfaction in Professor Cain's formulas. Hydraulic engineers who prefer simple formulas, simply arrived at, may be satisfied to use the writer's approximate theory of arch dams. Occasional check calculations for critical cases, for instance, when the stresses, calculated by the approximate method, appear to be near the permissible limits, may be made by the exact theory and without unduly lengthening the procedure.

The writer is unable to appreciate the efforts of Mr. Jakobsen in calculating the influence of shear on the deflections of the Salmon Creek Dam. In calculating the angle at the base of the dam, Mr. Jakobsen assumes the cantilever as carrying all the load for the full height of the dam, and the arches nothing, whereas apparently the opposite is more nearly correct in this particular case. Therefore, Mr. Jakobsen's results are evidently greatly in error.

In the second example, relating to the short cantilever below Elevation 1035, Mr. Jakobsen apparently overlooked the fact that the writer gave the value, 20 000 000 lb., only for the purpose of illustrating the impossibility of such a cantilever deflecting "elastically" the amount measured. The total water pressure on the 28-ft. cantilever is little more than 250 000 lb., or only about 1% of the load assumed by Mr. Jakobsen. His calculated shear deflection, therefore, is nearly one hundred times too large.

In calculating the probable size of the crack at the base of the Salmon Creek Dam, Mr. Jakobsen makes the assumption that the cantilever stands on its edge, which is absurd, considering the fact that the weight of the masonry above is nearly 700 000 lb. per lin. ft. of dam.

The writer is also unable to follow Mr. Jakobsen's reasoning, that an unreinforced concrete beam of the size considered in his example and for which standard calculations indicate tensile stresses of 2 750 lb. per sq. in., should not be cracked, but "shows that the ordinary bending formulas are entirely inapplicable."

There is little probability of seeing, from a distance, the cracks on the warped down-stream face of the Salmon Creek Dam, at either Elevation 1075 or Elevation 1155, and this may be the reason why such cracks, apparently have not been discovered. At the upper elevation, the dam is only 11 ft. thick, and there is no reason to assume that a beam of this thickness should act so greatly at variance with theory, as Mr. Jakobsen would have one to believe. The fact that apparently no one has seen those cracks is not conclusive proof of their non-existence, and if standard calculations show, theoretically, tension stresses 10 to 20 times larger than average concrete is known to be able to stand without

breaking, the conclusion as to the occurrence of cracks would appear to be well justified.

Mr. Dunn deserves much credit for having called attention in such an able manner to the probability of uneven loading of the horizontal arches at certain dam sites. In fact, the writer has had the same experience in similar cases, and for this reason has suggested* that "in addition to the foregoing investigation of the cross-section of the dam near the arch crowns, in general it will be desirable to investigate also the sections at about the quarter points [or any other points] of the highest arches, * * *." The writer has come to the conclusion that in all cases for which the difference of the arch load at the crown and near the abutments is not very large, the maximum load may be assumed as acting uniformly over the whole arch. Rib-shortening and shrinkage have the tendency to move the arch thrust at the abutments toward the air side of the arch, thus approaching again more closely the circular form of arch axis. However, every dam site has its own peculiarities and no definite rules can be stated.

Mr. Miller's excellent presentation of the derivation of exact formulas for secondary arch stresses is noteworthy, and his curves which show the overestimate of the value of the deformation thrust, H, for the various approximations, will be of good service to designers. Thus, it is possible to judge from those curves whether, for an arch of given dimensions, the error involved by the use of an approximate formula is large enough to warrant the application of a more accurate one; or one may use the approximate formulas and deduct the percentage of error, according to Mr. Miller's curves, to arrive at the exact values.

Mr. Hill questions whether the writer's formulas for cantilever deflections are strictly applicable to dams. Any theory applied to an unreinforced concrete structure which has developed cracks, is naturally more or less unreliable for such a state of deformation, because the statical conditions of the structure may have changed considerably at the instant that cracks occurred. If the formulas indicate excessive tension, and, therefore, the probability of cracks, it is not to be wondered if the actual deflections are out of all proportion to what pure theory would show them to be. It is hard to imagine a thick dam as simply supported ("hinged") without rather complex conditions resulting at the base from large deflections.

The discussion by Messrs. Williams and Greene is most interesting, and the hinged arch dam described therein is no doubt unique in the history of dam design. For the particular dam site, that is, in a gorge with a level bottom, the provision for a hinge at the base of the dam appears to be a good solution, inasmuch as a horizontal crack would probably have developed there in any case, because the span of the arches is too long as compared to the height of the dam.

As a general rule, it may be stated that, in order to enable elastical cantilever and arch action in unreinforced dams, the span of the highest arches

* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 25.

should not be much more than about twice the height of the dam.* Otherwise, the vertical cantilevers have to be anchored by steel bars to bed-rock or to enlarged horizontal cracks near the foundation.

It has been claimed by some engineers that the usual methods of calculating the deflections and stresses in cantilevers do not apply to the vertical cantilever slices of arch dams. For thin and comparatively low dams, there is no reason why the standard methods of structural design should not be applicable with the same degree of accuracy as in building or bridge construction. For instance, the Barren Jack Dam is only 5 ft. thick at the base and 2 ft. thick at the crest, and the concrete is probably of the same quality as that in other structures, so that the same assumptions and laws may be applied. It is possible, however, that, for very high and thick dams, the usual assumptions as to Hooke's law, modulus of elasticity, etc., may have to be modified somewhat. The question is, how far and for what size of dam are such new rules to be applied. This can only be determined by tests.

Some engineers also rely, to a great extent, on the "flow" of concrete under high compression, and on the "time factor". The writer would suggest caution in applying such assumptions indiscriminately and before definite rules have been established by tests on dams. In the present state of knowledge, it would appear to be safer practice to calculate the stresses in arch dams according to the well established theories and assumptions used in the design of other concrete structures, and then make, if desired, a correction for flow, time factor, etc., of 10, 20, or 50%, or whatever may be judged to be reasonable for the particular structure in question.

Cantilever and arch deflection stresses of considerable magnitude are likely to occur in any arch dam, and such stresses should be taken care of, in addition to those resulting from pure cylinder action. In no case should designers of arch dams continue to imitate the ostrich which is said to bury its head in the sand in the endeavor to hide from a threatening danger.

One of the main purposes of the writer's recent papers was to arouse among hydraulic engineers more general interest with regard to true arch action in arch dams. Instead of following strictly theoretical lines, as others had done, he chose to simplify the various problems encountered by developing approximate methods and formulas that might be more easily understood and more readily applied, without the necessity of delving into all the intricacies of the theory of the elastic arch. Several exact methods of designing arches according to scientific principles were developed a score or more years ago. One of the main reasons why such methods have not been used more generally for arch dams is their complicated form which makes it difficult for the average engineer to study and apply these methods in the usual short time allowed for the design of dams. The writer hopes, therefore, that his approximate formulas may be of help to engineers as an intermediate step on the ladder to perfection.

Other cases are known in structural design where approximations are used extensively for the sake of simplicity. The most striking example is the present standard design of continuous girders, for which the coefficients of

* See also "Beitrag zur Berechnung massiver Staumauern", by Prof. A. Rohn, *Schweiz. Bauzeitung*, March 11th, 1922, p. 128.

$\frac{1}{8}$, $\frac{1}{10}$, and $\frac{1}{12}$ are used for calculating the bending moments, instead of the theoretically more exact but much more complicated "three-moment equation" by Clapeyron. Engineers versed in structural design are well aware that the difference between the approximate and the theoretical results is often 50% and more, particularly in the case of heavy live load on continuous girders of uneven spans. However, the approximate formulas are generally used, on account of their simplicity.

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Paper No. 1485

THE FLOOD OF SEPTEMBER, 1921, AT SAN ANTONIO, TEXAS.*

By C. Terrell Bartlett,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles W. Sherman, Allen Hazen, Edgar Jadwin, C. E. Ellsworth, and C. Terrell Bartlett.

Synopsis

This paper describes the flood of September 9th, 1921, in the San Antonio River, at San Antonio, Tex., and in several tributary creeks all of which traverse that city, the fundamental conditions of topography and climate, and the meteorological factors and hydraulic elements of the flood in so far as it was possible to determine them.

Before describing in detail the recent flood experienced by the City of San Antonio, and its engineering aspects, it will be well to outline briefly the principal geographic, topographic, and climatic features of the region.

Geographically and geologically, the State of Texas is divided into three grand provinces, as shown in Fig. 1. The first is the Rio Grande Plain, or Coastal Plain, an alluvial prairie extending from the boundaries of Louisiana and Eastern Oklahoma to the Rio Grande, and from the Gulf of Mexico inland from 150 to 200 miles. The topography ranges from the coastal flats through gently undulating and rolling country to altitudes of 400, 600, and 1 000 ft., along the inner margin of this region as one follows it from the Red River south and west to the Rio Grande.

The second of these great provinces extends from the margins of the Coastal Plain west to the Pecos River, the New Mexico line, and, northward, it embraces the Panhandle. This is the Central Plain, and forms the southern extension of the great plains of the Dakotas, Kansas, and Eastern Colorado. The third division known as Trans-Pecos Texas is part of the Rocky Mountain region.

* Presented at the meeting of October 5th, 1921.

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The line of division between the Rio Grande Plain and the upland plain or plateau is marked sharply and definitely by a great geologic fault which runs south about 300 miles from the Red River on the northern boundary of the State and thence swings west for 150 miles, where it crosses the Rio Grande into Old Mexico. This fault is known as the "Balcones Escarpment", and is characterized by a sudden rise in altitude from the Coastal Plain to the plateau region. At Del Rio, where this escarpment crosses the Rio Grande, the upper plain is about 1 200 ft. higher than the Rio Grande Plain, and the rise is very abrupt. In the vicinity of Fort Worth, the elevation is not more than 300 ft. and rises in a gradual incline across 15 to 20 miles.

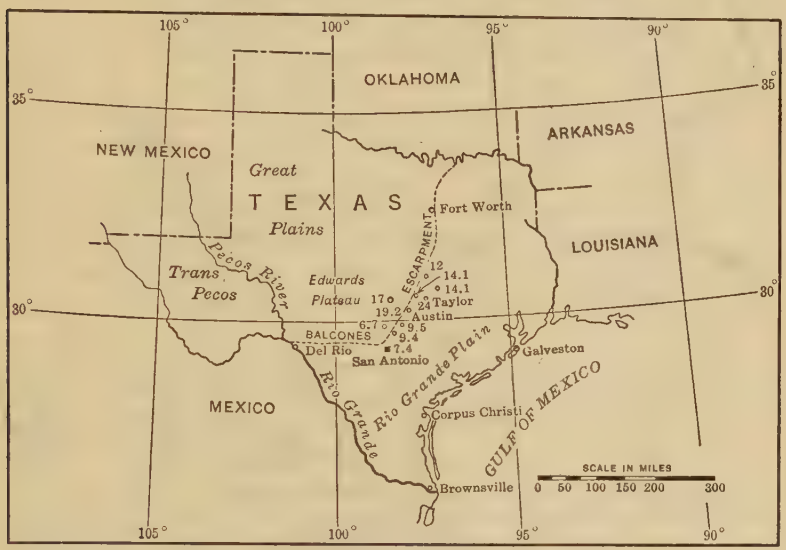


FIG. 1.

The mean annual rainfall of Eastern Texas along the Louisiana line is 50 in. Thence, westward, it decreases to 30 in. along the eastern margin of the plateau, or Central Plain, and westward and southward to 20 in. at Del Rio and Brownsville. Although the influence of altitude is not apparent in the mean annual rainfall of Texas, it has long been recognized that in many cases the sudden rise at the Balcones Escarpment causes intense precipitation along and just above its margin. Due to the proximity of the Gulf, intense rains are also frequent along the immediate coast.

San Antonio lies in the Rio Grande Plain immediately below the Balcones fault zone, adjacent to the great angle where the plateau juts out into the plain. From the northern limits of the city, the country rises to the northwest for about 12 miles to the elevation of the plateau, 700 ft. above the general level of the city. On this sloping margin of the plateau are three comparatively small water-sheds the streams of which traverse the city and unite near its southern limits, as shown in Fig. 2. All the streams were in severe flood on the night of September 9th, 1921.

The City of San Antonio occupies 36 sq. miles and comprises a central valley the elevation of which is 650 ft. above sea level. It is bordered on the east and north by ridges about 100 to 150 ft. above the center of the city, and on the southwest by a slightly elevated plain. By the census of 1920, the population was 162 000.

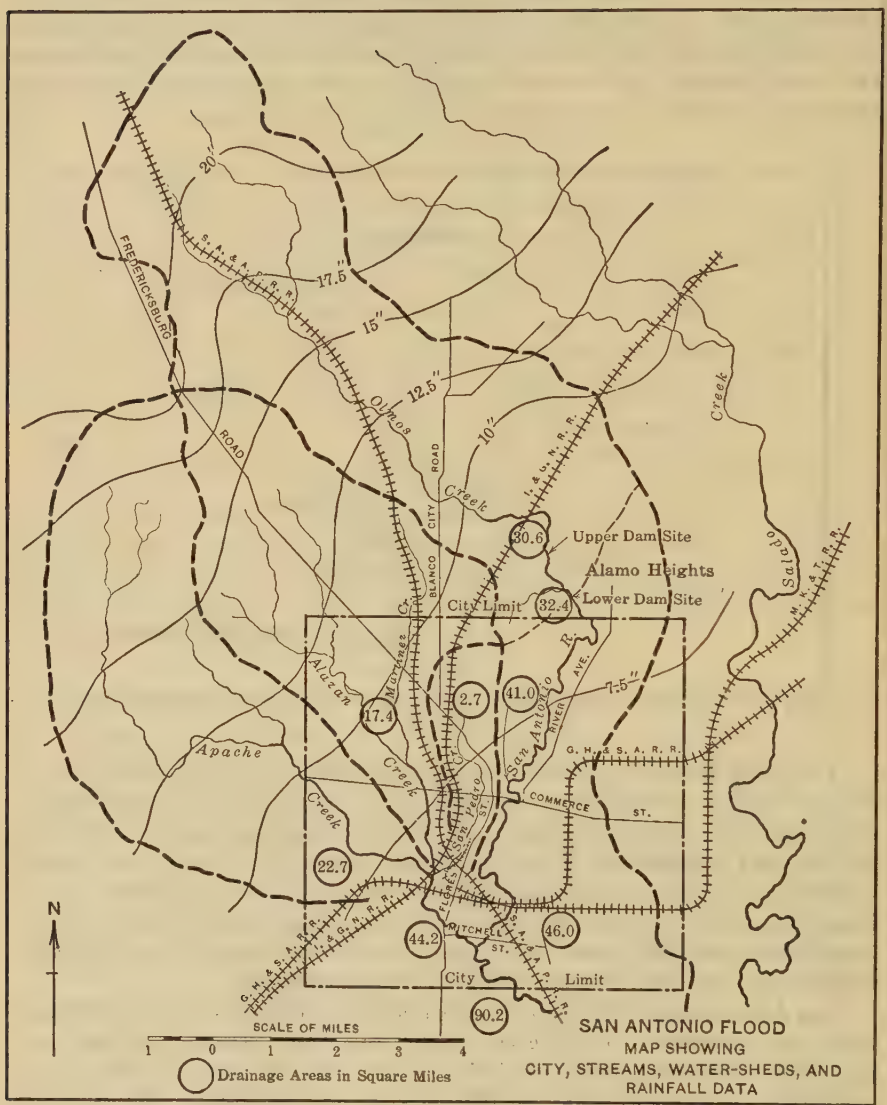


FIG. 2.

The San Antonio River rises from fissure springs near the northern city line, about a mile east of its center, and winds through the heart of the city within a well-defined flood-plain from $\frac{1}{2}$ to $\frac{3}{4}$ mile in width. The flow from

these springs ordinarily fluctuates between 50 and 150 sec-ft. The length of the valley within the city limits is 6.3 miles, and the length of the river channel is 11.9 miles, with a fall across the city of approximately 90 ft. The greater part of the business district, some of the industries, and the older and more thickly settled residential districts, lie in the immediate river valley.

The largest of these three water-sheds tributary to the San Antonio River lies north of the city, and is drained by a watercourse known as Olmos Creek which ordinarily is dry. The Olmos water-shed is 32.4 sq. miles in extent which, with the lower area tributary to the river proper, makes a total area of 41 sq. miles above the center of the city.

Within the city and 2 miles north of its center, San Pedro Creek rises from springs the flow of which is seldom greater than 10 sec-ft. Above these springs is a small water-shed entirely within the corporate limits.

Above the successive confluences of Alazan and Apache Creeks, the San Pedro Creek has a tributary area of only 2.7 sq. miles, but when augmented by these watercourses, it has a total drainage area of 44.2 sq. miles where it empties into the San Antonio River near the southern limits of the city.

Both the Alazan and Apache Creeks are dry arroyos. The former enters the city from the northwest and has a water-shed of 17.4 sq. miles. The latter enters from the west and has a water-shed of 22.7 sq. miles, measured, in both cases, to their junctions with San Pedro Creek about 1½ miles south of the center of the city.

The Lower San Pedro traverses the western part of the business district, and, together with the Alazan and Apache Creeks, runs through some of the cheaper and more thickly settled residential districts, especially the Mexican quarter.

A drouth of several months' duration was broken on the night of Thursday, September 8th, 1921. San Antonio and the region to the north received a soaking rain, followed by occasional hard showers, during the day on Friday, September 9th.

Owing to the parched condition of the ground, no run-off occurred from the rain of Thursday night and only a little, except from areas within the city, from the day rain of Friday. The day rains of Friday were followed by a severe electrical storm, continuing from 6 P. M. to 9 P. M., during which the region north and northwest of the city was visited by intense rainfall which continued with reduced severity until 11:30 P. M.

Olmos Creek, north of the city, overflowed its banks and swept into the city about 10 P. M., reaching its crest within an hour. At the center of the city, the river, not more than 2 ft. above normal at 6 P. M., rose with increasing rapidity, overflowing its banks immediately north of the business district at midnight. Within an hour, a large part of the business quarter was inundated, and the crest was reached about 2 A. M. At this time, six of the principal north and south streets were channels carrying the swift flood waters across a great bend of the river. The water in these channels ranged in depth from 1 to 8 ft. where they crossed Houston Street, the principal east and west street, and at places in the business area the water was 12 ft. deep. North of

Houston Street, the water spilled through five streets over a low divide to the west into the Upper San Pedro Basin.

In the central district, the river rose from 2 to 8 ft. above all bridge floors and at each bridge a great mass of débris was caught by the piers and railings.

In the northern part of the city, the rush of water in the meantime had inundated Breckenridge Park along the river and all the adjoining low residential and industrial areas. Hundreds of homes were flooded to depths varying from a few inches to 8 and 10 ft. In this section, a few poorly built frame dwellings were washed away. A scum of heavy fuel oil from some of the industries coated everything it touched, and, incidentally, everywhere below left clear and unmistakable high-water marks. In the business district, all the basements were filled, and most of the first floors were flooded to depths of from a few inches to 12 ft.

The severity of the flood crest in the business district was undoubtedly somewhat mitigated by the pondage of water and its retention by houses and other obstructions in the valley above. In like manner, the obstruction formed by the business district, the storage of water in basements, and probably, also, a considerable absorption from the river channel into the gravel beds which underlie many parts of the valley, reduced the flood markedly below. This effect was partly counteracted as far as the flooded residence district just below the business section was concerned, due to two low dams and the blocking of two railway trestles with drift, all of which were situated about 1 mile below the center of the city. Below these obstructions, the flood was carried by the river channel and natural overflow bottoms which have not been encroached on by retaining walls and buildings to the same extent as in the central and upper parts of the valley. Notwithstanding many narrow escapes, there was practically no loss of life in the valley of the San Antonio River.

Owing to the comparatively light rainfall in the city, the small Upper San Pedro Basin did not flood badly, and its crest passed before the overflow from the San Antonio River. The latter inundated a number of buildings in the western edge of the business district, the water in it and the San Pedro forming a continuous sheet.

Alazan Creek, with a comparatively steep grade from the high ground northwest of the city, and its two branches flooding simultaneously, swept suddenly through the thickly settled low areas along its banks, playing havoc with bridges and tearing loose and destroying many cheap frame houses. It was in this section that the principal loss of life occurred. Conditions on Apache Creek were similar, but there was less development along its banks and the loss was smaller. Below the confluence of Alazan and Apache Creeks, the San Pedro again passes through a thickly settled area where there was considerable damage and loss of life. The total certain loss of life was 52, with 23 missing, most of whom were probably lost, making a total of 75.

The aggregate property damage was between $5 000 000 and $10 000 000. The greatest damage was to merchandise in flooded basements and on first floors. Next in amount was the loss in homes to clothing, furniture, and to the finish of walls and interiors from the muddy, oil-laden waters. The loss to the municipality was heavy, principally to pavements and wooden bridges

FIG. 3.—RIVER CHANNEL THROUGH HEART OF CITY.

FIG. 4.—RIVER CHANNEL THROUGH HEART OF CITY, DURING THE FLOOD.

FIG. 5.—SOUTH ALAMO STREET BRIDGE.

destroyed or injured, together with the expense of sanitary measures and for cleaning up the débris. One three-span concrete girder was destroyed by scour.

In general, substantial buildings were little damaged structurally, although there was a heavy loss of plate glass broken by water pressure and by drift, and, in total, a heavy damage for refinishing. The public service corporations suffered some loss, the principal item, however, being in decreased earnings for several days.

An interesting feature was the continuous low-pressure water service to the greater part of the city by the direct head of the artesian wells. An incident of engineering interest was the loss of practically all flooded wood block pavements which simply floated off. A large percentage of these blocks were gathered up, however, and relaid.

Several concrete basement floors were burst upward by pressure from water which traveled underneath, through gravel beds, before the basements were flooded from the top. On the first floor of buildings, tile floors, in several instances, were cracked loose at the joints, and the first-floor concrete slabs were undoubtedly severely strained by pressure from beneath. This occurred where the basements were flooded and the water in the streets stood from 2 to 3 ft. higher than inside the closed doors where the occupants were attempting to prevent its entrance.

The rain storm which caused the San Antonio flood was typical in that it broke on the margin of the Edwards Plateau. Rainfall data over the water-sheds above San Antonio have been secured by the writer and by the Engineer Corps of the Army. These data are based on records at about twenty points; about one-third were secured by standard rain gauges and one-fourth by measurements, in straight-sided cans, taken by farmers who keep more or less regular records. The remaining readings were secured by reducing measurements made in various classes of receptacles.

The testimony is so general as to the severity of the rain along the rocky rim of the Olmos water-shed and farther north in the plateau that there can be no question that at points the total precipitation was in excess of 20 in. Fig. 2 shows the total rainfall for the entire storm, from 6 P. M., Thursday, to midnight on Friday, a period of 30 hours. For the Olmos water-shed, the average rainfall was 14 in. and for that of San Antonio proper, below the city limits, it was 7.5 in., a mean depth of 12.5 in. for the entire basin.

The general storm over Central Texas was heavy, not only above San Antonio, but also at San Marcos and Austin, along the edge of the plateau. It is significant, however, that the heaviest rain, 19.5 in. in 12 hours or 23.1 in. in 24 hours, fell at Taylor, which is neither close to the coast nor adjacent to the plateau, but well out in the lower plain where previous statistics indicated that all the probabilities were against a rain exceeding 10 in. in 12 hours or 14 in. in 24 hours.

The available rainfall records of the general storm have been indicated on Fig. 1.

The greatest hourly intensities in the San Antonio storm were between 6 P. M. and 11:30 P. M. During this 5.5 hours, the rainfall ranged from 1.56 in. at the San Antonio Weather Bureau to 3.5 in., 4.5 in., 6.2 in., and possibly as

high as 15 in., at various points. The average on the water-shed for this 5.5 hours has been placed at 6.5 in.

The Engineering Corps of the writer's firm has made a hydrographic survey, as careful as circumstances permitted, of the slopes and areas of the flood crest at various points. It appears that ½ mile above the city limits, the flood peak in the Olmos was between 31 000 sec-ft. as a minimum, and 39 000 sec-ft. as a maximum, from 32.4 sq. miles. This represents from 960 to 1 200 sec-ft. per square mile. The peak discharge at the center of the city is estimated at 23 700 sec-ft., or 580 sec-ft. per square mile, from 41 sq. miles. Below the center of the city, 1½ miles, the crest is estimated at 1 500 sec-ft., or 333 sec-ft. per square mile, from 45 sq. miles.

The rapid reduction in flood crest both in total and in second-feet per square mile is significant in that the reduction was very much more rapid with increased area than would be indicated by the usual curves for second-feet per square mile. This was doubtless due in part to the distribution of the rainfall, but the influence of the river flood-plain in impounding the flow was also a marked factor. The influence of buildings probably increased somewhat this retardation over the natural conditions of a timbered flood-plain, although the streets were more efficient than natural channels. The average slope of the Olmos Valley above the city is 25 ft. per mile, the average fall of the river valley across the city being 14 ft. per mile.

One feature of the flood that will be difficult for Eastern engineers to understand is the comparatively small run-off. Above the city, the Olmos was not out of its banks more than 6 hours, the crest falling almost immediately. The flow increased from about 4 000 to 35 000 sec-ft. and fell again to 4 000 sec-ft., in 6 hours. At the center of the city, the flow ranged from 4 000 to 23 700 sec-ft. and back to 4 000 sec-ft., in about 11 hours.

From unofficial gaugings taken by the U. S. Geological Survey on the day following the flood, and by gaugings of the water-stage register, and from other data, a rough computation has been made of the run-off for the storm for a period of 7 days, which indicates that it did not exceed 10 000 acre-ft. from an area of 42 sq. miles above the station. All the storm flow had practically run off within the period of one week. It is unfortunate that the water-stage register of the Geological Survey was located at the South Alamo Street concrete bridge which failed. Although the gauge itself was undamaged and recorded the levels, except for 3 hours at the crest, the failure of the bridge interfered with the channel so that no accurate rating curve can be established for the flood conditions.

This estimated run-off of 10 000 acre-ft. is 36.4% of the depth of rainfall for the entire storm. However, by far the greater portion of this run-off was from 32.4 sq. miles above the city. It is probable, in fact, that the run-off was as great from the upper basin as from the entire basin, the absorption of flood-waters by gravel beds in the lower valley probably amounting to as much as the run-off from the lighter rain on the lower 9 sq. miles within the city.

Assuming that the run-off from the upper basin of 32.4 sq. miles was as great as at South Alamo Street, 5 miles down stream, that is, 10 000 acre-ft.,

Fig. 6.—St. Mary's Street Bridge.

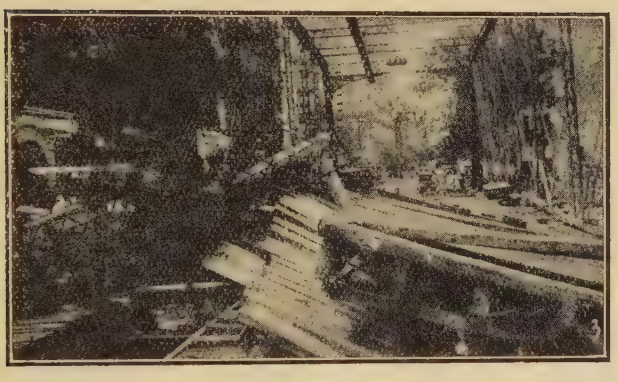


Fig. 7.—Looking North on Navarro Street Bridge.

Fig. 8.—Looking Toward Business District from Pecan Street.

according to the estimate, the run-off from the Olmos Basin was about 42 per cent. Regarding the Thursday night and Friday rains, amounting to an average of 6.5 in., as producing only 10% of the actual run-off, there would have occurred an average run-off of not more than 70% from the 5.5-hour rain of Friday night falling on pre-saturated ground and amounting to from 3.5 to 15 in.

The flood peak on the Upper San Pedro Creek was not excessive either from the run-off of the earlier local storm, or from the later overflow from the river. The maximum discharge was possibly 1 500 sec-ft. from 2 sq. miles.

On the Alazan Creek the water-shed of which is ideal for a quick flood concentration and on which a small earth dam was washed out, the peak flow was approximately 33 000 sec-ft. from 16.9 sq. miles, or at a rate of 1 950 sec-ft. per square mile.

The Apache Creek is estimated to have flowed at a rate of 15 500 sec-ft. from 22 sq. miles, or 704 sec-ft. per square mile.

The average rainfall on the water-sheds of these two creeks was probably as high as that on the Olmos Basin and greater than that for the combined Olmos-San Antonio Basins. The flood peaks from the creeks reached the San Antonio River before its own crest so that although the flood experienced in the lower river was of unusual proportions, it was not of the extreme character suffered in the individual streams above their junction near the southern limits of the city.

San Antonio has been subjected to floods from its river and the western creeks at intervals of from 8 to 15 years. The last damaging floods occurred twice in 1913 and once in 1914. Spanish records tell of a flood in July, 1819, which was undoubtedly somewhat greater than the recent one. As a result of the floods of 1913-14, the city engaged Leonard Metcalf and H. P. Eddy, Members, Am. Soc. C. E., to report on methods of flood prevention, and their report filed in December, 1920, foretold the possibility of just such a flood as that which occurred.

As a result of the flood of 1921, a committee of local citizens and engineers was formed to study the conditions and make immediate recommendations on flood prevention, and it is probable that extensive works to that end will be undertaken.

The writer wishes to express his appreciation of valuable information secured from data collected in the report by Messrs. Metcalf and Eddy, to C. E. Ellsworth, Assoc. M. Am. Soc. C. E., of the U. S. Geological Survey, for advance stream records, and to Col. Edgar Jadwin, Corps of Engineers, U. S. A., for rainfall data and the opportunity of comparing discharge estimates.

DISCUSSION

CHARLES W. SHERMAN,* M. AM. SOC. C. E.—The information contained in this paper is of especial interest to the speaker because of his intimate connection with the preparation of the Metcalf and Eddy report to which Mr. Bartlett has referred.

This report embodied the results of a special study of local conditions, including such information relating to previous floods in San Antonio River as could be obtained, and a detailed examination of the records of intense rains in Texas and of the flood discharge of streams throughout North America.

As Mr. Bartlett has stated, floods which did considerable damage along the San Antonio River, occurred twice in 1913, and one which just escaped doing material damage occurred in 1914. Reasonably satisfactory information regarding these floods was obtainable, but data on earlier floods were extremely fragmentary and unsatisfactory. The dates of their occurrence and some approximation as to the height of the flood water at a few points were all that could be obtained, and much of this information did not become available until the investigation was nearly completed. Therefore, the estimation of the magnitude and frequency of serious floods had to be made on very incomplete information, and estimates of possible floods based on rainfall records proved of material significance in reaching the final conclusion.

The report recommended flood-control works for the San Antonio River, including an improved river channel with a capacity of 12 000 cu. ft. per sec. and a detention basin above the city of sufficient size to retain an additional 10 000 cu. ft. per sec., so that the complete works would care for a flood of approximately 22 000 cu. ft. per sec., or nearly three times the flood flow in 1913. This figure seems to have been very nearly the maximum rate of flow in the river during the flood of September, 1921.

The flood of 1913 was so much in excess of any other flood within the memory of the people of San Antonio that provision for a 50% greater flow in the river channel probably seemed to them to be adequate for the protection of the city. If the flood of 1921 had not occurred, local sentiment might not have followed the recommendations of the report to the extent of including the detention basin in the works to be constructed, although it is more or less probable that gradual improvement of the river channel to a capacity approximating that recommended in the report would have been carried out.

In view of the scarcity and unsatisfactory character of the information available, the conclusions of the report seem to be almost inspired prophecy, and it may be permissible to quote a few sentences showing how these conclusions were promptly and disastrously verified by the flood of September, 1921. The report states:

"We doubt if the citizens realize the ruinous loss which would result to-day with the present condition of the river channels, from such a flood as that of a century ago (1819). When such a flood will recur, no man can say. But that it will recur is certain. * * * We counsel the wisdom of pushing

* Cons. Engr. (Metcalf and Eddy), Boston, Mass.

this work, however, while the memory of recent floods is vivid, lest the public mind relapse into inaction in a false sense of security, only to be startled later by stern reality when the inevitable flood shall come. We urge that your citizens shall remember that this disastrous flood is just as likely to come next year as at any other time."

Remembering that this report was made in December, 1920, it appears that the prediction was verified within nine months of its presentation.

A report from the local Governor to the Provincial Governor, accounting for damages to Government property by the flood of 1819, appears in the county records. Texas was then a part of Mexico, which itself was a Spanish Colony. The property of certain rebels against the Government had been confiscated, including several houses of adobe construction. The flood water extended around some of these houses, which resulted in the softening of the walls so that the houses collapsed. It is impossible to say whether the water was 1 ft. or several feet deep, and there seems to be no information available regarding the condition of the stream channel. It is known that irrigation was practiced extensively, and it is possible that the diverting dams may have had considerable effect in raising the water surface. It is also probable that whatever bridges existed were of wooden trestle construction and may have seriously obstructed the flow of the stream. Exact knowledge of the flood height, therefore, would be insufficient as a basis for a satisfactory estimate of flood discharge. The report concluded that such discharge might have been between 20 000 and 30 000 cu. ft. per sec. Mr. Bartlett's conclusion is that the flood of 1819 was probably somewhat greater than that of 1921.

It is not impossible that the records of the old Spanish Missions, the earliest of which was founded about 1720, might also contain information of great significance in this connection. The speaker, however, does not know whether such records exist.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper is a description of a most interesting flood. The writer had occasion to visit San Antonio shortly after the flood, and he read the paper carefully in connection with his own observations and commends the author for a concise and accurate statement.

In this case, the reason for anticipating a large flood had been pointed out professionally to the city by Messrs. Metcalf and Eddy, who had studied the conditions only a short time before. The lesson to be learned from this and other floods is that growing cities must give more attention to flood channels that are being closed by bridges, buildings, and encroachments of all kinds.

The San Antonio flood and other floods sometimes go so far beyond previous experience as to suggest that there is no law of flood frequencies, that governs. The writer does not believe that this view can be supported.

Engineers are learning more every year about the laws that govern the frequency of floods. Each new record helps. It may be expected that the continuous records of gauging stations will ultimately be more useful in establishing these laws than records of destructive floods that occur now and then where there is no continuous gauging. These extreme floods serve

* Cons. Engr. (Hazen, Whipple and Fuller), New York City.

to support and emphasize the conclusions that may be deduced from the study of records from regular gauging stations that much larger floods are to be logically anticipated at infrequent intervals.

There is a flood to be expected each year. There is a greater flood to be expected on an average once in 10 years, and there is a still greater flood to be expected on an average once in 100 years, and so on, and with innumerable intermediate steps. The chance is only one-hundredth of 1% that the 10 000-year flood will occur at any one place in any one year, but when the whole country is considered, some cities will suffer from such floods with a frequency well within the experience of human life, and the possibility of their occurrence at any place in any year must be taken into account.

The writer hopes that some day there will be evolved a much better basis for estimating these probabilities and for determining how much may be done in preventing them without, on the other hand, going to unreasonable extremes.

EDGAR JADWIN,* ESQ. (by letter).—This paper contains an excellent account of the San Antonio storm and its relation to the general terrain, as well as to the local conditions, particularly as it was written soon after the storm and probably before all possible data could be obtained and digested. The records of the U. S. Engineer Office at Fort Sam Houston agree substantially with those presented by the author and only a few slight differences are worthy of mention.

Some slight differences in data were observed regarding the flow in the various streams, which are submitted herewith, as they show a slightly more constant maximum discharge per square mile. The maximum discharge from the Olmos, as computed by the office of the Engineer, Eighth Corps Area, was 25 000 sec-ft., which would give an estimated flow of this basin of about 930 sec-ft. per sq. mile. For Apache Creek, an estimate fixed the maximum discharge at 22 560, or 947 sec-ft. per sq. mile. On the Martinez-Alazan, the maximum flow estimated was 24 000 sec-ft., or 1 223 per sq. mile.

The flood was the result of a rain storm, of which the zone of greatest precipitation lay easterly and westerly across the head-waters of these streams. In San Antonio, the flood peaks of the Alazan and Apache Creeks arrived at different times. The peaks of both had passed before the slower gathering flood from the San Antonio River arrived. Had a combination of circumstances made the crests of the various creeks arrive simultaneously, the damage to the lower part of the city undoubtedly would have been much greater. As it was, the river, in the southernmost part of the city, overflowed its banks only slightly.

Since his paper was written, the committee of engineers referred to by Mr. Bartlett, has recommended methods to prevent a future recurrence of these floods. This plan provides that the channels of the San Pedro, Alazan, and Apache Creeks be widened, deepened, and straightened, that the grades along the banks generally be raised, and that a strip of varying width on these streams be reserved through the town. This part of the city is not nearly

* Col., Corps of Engrs., U. S. A., Fort Sam Houston, Tex.

as thickly settled and the land is less valuable than that in the valley of the San Antonio.

On the San Antonio, where the land is too valuable to allow purchase of a strip wide enough to carry the full flood flow, the Committee recommends a straightening of the river by the elimination of several large bends within the city and a slight deepening and widening of the channel. This channel now has a capacity of approximately 6 000 sec-ft. in its improved parts, and will have its capacity increased to 15 000 sec-ft. In addition, the Committee recommends constructing a concrete retention dam to form a dry reservoir at a point near the mouth of the Olmos and about 2 000 ft. north of the northern boundary of the city, where borings indicate a good foundation. This dam is to be provided with permanent openings to allow the discharge of 6 000 sec-ft., retaining all peak water in excess of that quantity until the channel can carry it. The estimated cost of this construction is $5 500 000.

C. E. Ellsworth,* Assoc. M. Am. Soc. C. E. (by letter).—The author is to be commended for his initiative and promptness in collecting much valuable data regarding the San Antonio flood and for his service to the Profession in making such data available.

The writer has prepared this discussion for two purposes:

1.—To make available in printed form, in advance of official publication, the records of stage and discharge of San Antonio River, that are on file in the office of the U. S. Geological Survey and the State Board of Water Engineers at Austin, Tex.

2.—To prevent the impression that a reasonably accurate rating and determination of discharge during the flood could not be made, because the water-stage register, referred to by the author, was located at the South Alamo Street Bridge, which failed.

In discussing the accuracy with which the discharge could be determined, the writer is unable to recall the exact statements which he made to the author at different times, immediately after the flood, but he was then of the opinion and probably conveyed the idea that the rating was less accurate than the final analysis indicates.

Like practically all such determinations during great floods, the calculations of maximum discharge and run-off are liable to considerable error. The writer believes, however, that the accuracy with which this flood has been measured is relatively high, because a practically complete graph record of stage was obtained and because the stretch of channel a short distance down stream from the gauge was relatively favorable for the determination of peak discharge by the slope method. Sufficient current-meter measurements of discharge are available for the low and the medium stages, so that no gross errors are possible in the rating. (See Table 1.)

The author has stated that:

"It is unfortunate that the water-stage register of the Geological Survey was located at the South Alamo Street concrete bridge which failed. Although the gauge itself was undamaged and recorded the levels, except for 3 hours

* Dist. Engr., U. S. Geological Survey, Austin, Tex.

at the crest, the failure of the bridge interfered with the channel, so that no accurate rating curve can be established for the flood conditions."

TABLE 1.—LIST OF DISCHARGE MEASUREMENTS OF SAN ANTONIO RIVER, AT SAN ANTONIO, TEX.*

Number of measurement.	Date.	Gauge height, in feet.	Discharge, in second-feet.	Method.	Location.
81	March 11, 1921	2.51	391	Current-meter	At gauge
87	Sept. 8, "	1.38	76	"	"
88	Sept. 9, "	3.31	622	"	"
89	Sept. 10, " †	20.14	15 300	Slope	2 000 ft. below gauge
90	Sept. 10, "	8.94	2 200	Current-meter	Nueva Street Bridge, 1.8 miles above gauge,
91	Sept. 10, "	5.57	866	"	Railroad bridge, 400 ft. above gauge
92	Sept. 21, "	3.13	153	"	At gauge
93	Oct. 4, "	2.90	117	"	"

*Numerous discharge measurements, not listed, have been made at stages less than 2.00 ft.
†Measurements of cross-section and slope were made on September 15th to 17th, 1921.

The water-stage register was installed in April, 1920, by the Engineering Department of the City of San Antonio, according to a plan prepared by the Water Resources Branch of the U. S. Geological Survey which maintains a District Office at Austin in co-operation with the State Board of Water Engineers. The site, at the down-stream side of the right abutment of the South Alamo Street Bridge (Fig. 5), was selected by Hans Helland, M. Am. Soc. C. E., then City Engineer, and the writer, after a thorough examination of the course of the river through the city and a careful consideration of all conditions that might affect the accuracy of gauge records and the determination of discharge. The writer is now more firmly convinced than ever that a better record of the flood could not have been obtained at any other location, except, possibly, farther down stream, in the vicinity of the section used for the determination of discharge by the slope method. The use of that section, however, is dangerous owing to the possibility of back-water from San Pedro Creek and from débris that is likely to collect at the crossing of the Missouri, Kansas and Texas Railroad Bridge. A study of the overflowed area and of the condition of the channel, due to obstructions caused by débris collected at bridge piers and other constricted sections, shows that the relation between stage and discharge at any section above the South Alamo Street Bridge would have been decidedly uncertain at any time during the flood, and that the flow of water below the gauge was relatively unobstructed and permanent. Even after the bridge had failed and partly collapsed, the channel capacity was not reduced as much as it was at many other bridges which did not fail, but at which greater quantities of débris were collected. Practically, the only effect the failure of the bridge had on the rating was caused by a few small blocks of concrete, which broke loose and lodged in the channel below the gauge. That effect has been determined quite definitely by comparison of measurements made before and after the flood, at relatively low stages. (See Fig. 9.) The difference in discharge between the curves, considered as a per-

centage of total discharge, would be less for higher stages and would be negligible at the crest; therefore, in the absence of definite information as to whether the blocks of concrete found in the channel after the flood were dropped there before or after the time of the maximum stage, both curves have been drawn through Measurement No. 89, which represents the discharge at the maximum stage.

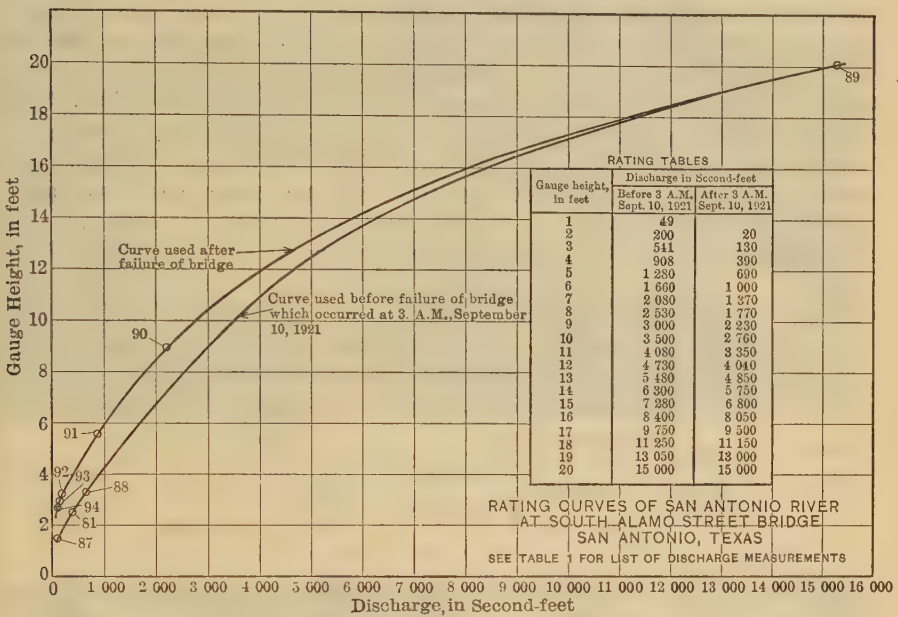

Gauge Height, in feet
Discharge, in Second-feet
Curve used after failure of bridge
Curve used before failure of bridge which occurred at 3 A.M., September 10, 1921
RATING TABLES
Discharge in Second-feet
Gauge height, in feet
Before 3 A.M. Sept. 10, 1921
After 3 A.M. Sept. 10, 1921
1 49
2 200 20
3 541 130
4 908 390
5 1 280 690
6 1 660 1 000
7 2 080 1 370
8 2 530 1 770
9 3 000 2 230
10 3 500 2 760
11 4 080 3 350
12 4 730 4 040
13 5 480 4 850
14 6 300 5 750
15 7 280 6 800
16 8 400 8 050
17 9 750 9 500
18 11 250 11 150
19 13 050 13 000
20 15 000 15 000
RATING CURVES OF SAN ANTONIO RIVER AT SOUTH ALAMO STREET BRIDGE SAN ANTONIO, TEXAS
SEE TABLE 1 FOR LIST OF DISCHARGE MEASUREMENTS

FIG. 9.

The discharge at the maximum stage was determined by the slope method, by using Kutter's formula. The high-water slope had been well marked by floating oil. A stretch of straight channel, 800 ft. long, about 2 000 ft. down stream from the gauge, was selected. The center of this stretch is near the intersection of Temple Street with the river. Cross-sections were taken

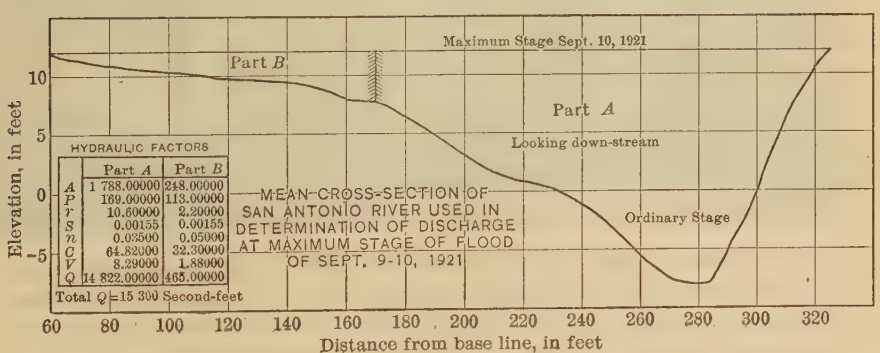


FIG. 10.

every 100 ft., and the slope was carefully measured on both banks. The slope on the left bank, for the upper 375 ft. and the lower 50 ft. of the stretch, was irregular and differed in form from that on the right bank. The left bank which is bordered by buildings and other obstructions, was overflowed, but the right bank which is high, was not overflowed. The difference was probably due to these facts. For the intervening 375 ft., the slopes on the two banks were in close agreement, and the mean slope was used, the area being the mean of the area at stations at 400, 500, 600, and 700 ft. below the upper end. Fig. 10 shows the mean cross-section and a tabulation of the hydraulic factors. In making the computations, the cross-section was divided into two parts, because of different conditions of roughness and mean depth. The channel in the slope section is straight and relatively clean and unobstructed, as compared with other parts of the river. Fig. 11 is a view of the channel looking up stream from a point on the right bank just below the lower end of the slope stretch, and Fig. 12 is a view from the right bank near the center of the slope stretch, showing the condition of the left bank and the flood line on it.

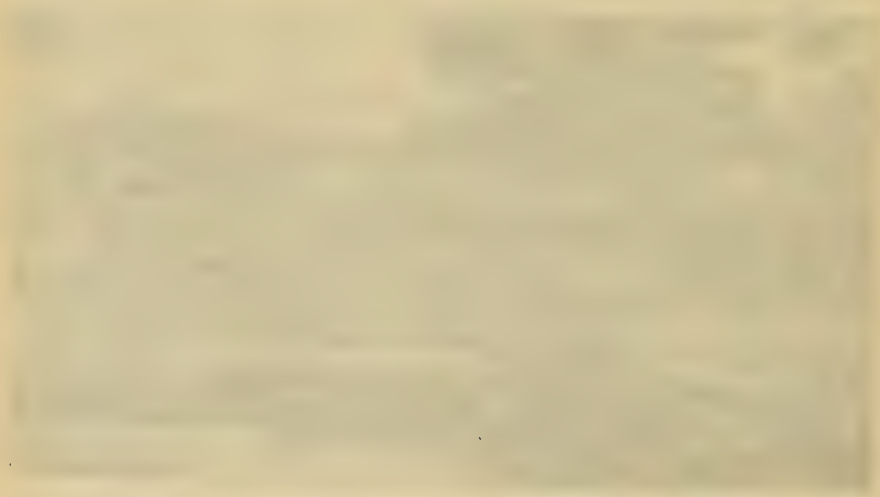
TABLE 2.—Gauge Height and Discharge of San Antonio River, at South Alamo Street Bridge, San Antonio, Tex., during Flood of September 9th-10th, 1921.

Hour.	September 9th 1921:			September 10th, 1921:		
	Gauge height, in feet.	Discharge, in second-feet.	Run-off, in acre-feet.	Gauge height, in feet.	Discharge, in second-feet.	Run-off, in acre-feet.
12-1 A. M.	1.30	68	6	11.93	4 690	388
1-2 "	1.45	80	7	14.30	6 040	499
2-3 "	1.92	176	15	18.50	12 100	1 000
3-4 "	2.52	369	30	19.40	13 800	1 140
4-5 "	2.98	534	44	17.49	10 300	851
5-6 "	3.13	588	49	15.40	7 300	603
6-7 "	3.29	645	53	13.38	5 190	429
7-8 "	3.44	701	58	11.61	3 760	311
8-9 "	3.18	606	50	9.95	2 740	226
9-10 "	2.92	512	42	8.72	2 090	173
10-11 "	2.67	422	35	7.67	1 640	136
11-12 "	2.68	426	35	6.40	1 140	94
12-1 P. M.	2.74	447	37	5.53	849	70
1-2 "	2.60	397	33	5.17	741	61
2-3 "	2.93	516	43	4.92	666	55
3-4 "	3.23	624	52	4.59	567	47
4-5 "	3.07	566	47	4.36	498	41
5-6 "	3.10	577	48	4.15	435	36
6-7 "	3.08	570	47	3.95	375	31
7-8 "	2.96	527	44	3.83	339	28
8-9 "	3.87	860	71	3.78	324	27
9-10 "	5.67	1 530	126	3.72	306	25
10-11 "	7.78	2 430	201	3.67	291	24
11-12 "	10.02	3 510	290	3.63	279	23
Mean.....	3.44	737	....	8.59	3 190	.....
Total.....	....	....	1 463	....	.....	6 318

Fig. 13 shows a graph of the gauge heights of San Antonio River and San Pedro Creek. The water-stage recorder on San Antonio River did not operate when the stage was above 15 ft., during the period from 1:45 to 5:45 A. M., on September 10th, 1921. The estimated part of the graph, however,

FIG. 11.—VIEW OF SLOPE STRETCH USED IN DETERMINATION OF PEAK DISCHARGE, LOOKING UP STREAM.

FIG. 12.—VIEW FROM RIGHT BANK NEAR CENTER OF SLOPE STRETCH.



can not be greatly in error, because the maximum height of 20.14 ft. is clearly marked in the gauge house, and the time it occurred has been quite definitely established as 3 A. M., by statements of residents living near the gauge, who also have stated that the crest stage lasted about 30 min.

Table 2 which gives the mean hourly gauge height, discharge, and run-off, in acre-feet, from 12 A. M., September 9th, several hours before the flood began, to 12 P. M., September 10th, when the river had receded to nearly normal stage, shows that the most disastrous flood in the history of San Antonio was caused by less than 8 000 acre-ft. of water.

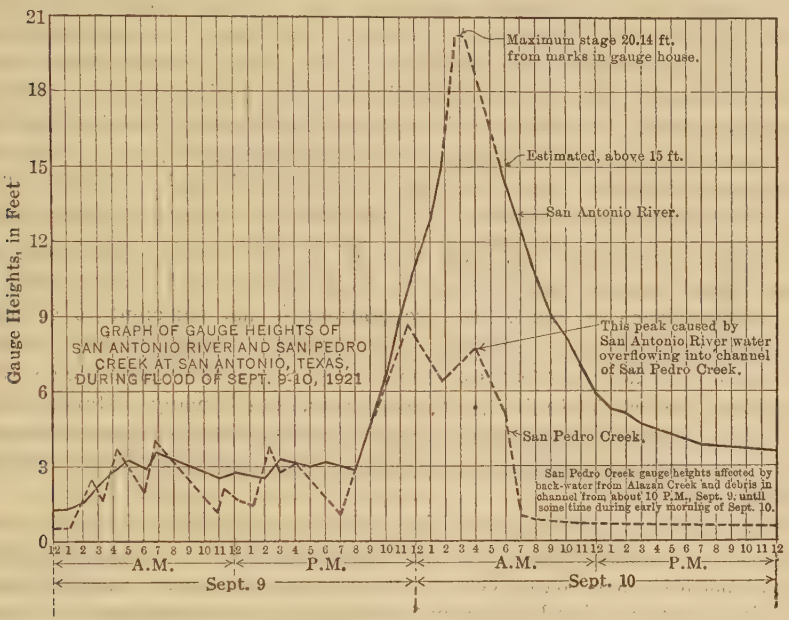

Gauge Heights, in Feet
21
18
15
12
9
6
3
0
Maximum stage 20.14 ft. from marks in gauge house.
Estimated, above 15 ft.
San Antonio River.
This peak caused by San Antonio River water overflowing into channel of San Pedro Creek.
San Pedro Creek.
San Pedro Creek gauge heights affected by back-water from Alazan Creek and debris in channel from about 10 P.M., Sept. 9, until some time during early morning of Sept. 10.
GRAPH OF GAUGE HEIGHTS OF SAN ANTONIO RIVER AND SAN PEDRO CREEK AT SAN ANTONIO, TEXAS, DURING FLOOD OF SEPT. 9-10, 1921
A.M.
P.M.
A.M.
P.M.
Sept. 9
Sept. 10

FIG. 13.

The water-stage register, on San Pedro Creek, is at the lower end of the covered part of the concrete-lined channel that extends from Durant Street to Arsenal Street. The graph of the gauge heights of San Pedro Creek is shown in connection with those of the San Antonio River, because of its possible value in determining the volume of water that passed over the divide from San Antonio River into the San Pedro Channel. The rise, shown in Fig. 13 as coming from San Antonio River, obviously did not come from any other source, as the flood-water from the San Pedro Basin had passed the gauge several hours earlier. It is impossible to compute with reasonable accuracy the discharge of San Pedro Creek during the flood, because of an indeterminate volume of back-water from Alazan Creek. The following rating table for the San Pedro gauge is submitted for the benefit of any one who may be sufficiently interested to attempt to compute the volume of water that passed from the San Antonio River Basin down the San Pedro Channel:

Gauge height, in feet.	Discharge, in second-feet.	Gauge height, in feet.	Discharge, in second-feet.
0.5	8	4.5	610
1.0	38	5.0	728
1.5	80	5.5	848
2.0	137	6.0	970
2.5	210	6.5	1 095
3.0	297	7.0	1 220
3.5	394	7.5	1 345
4.0	500	8.0	1 470

This rating table is directly applicable only at such times as back-water did not affect the San Pedro gauge heights. The time of the beginning and the ending of back-water is not definitely known, but it probably began about 10 P. M., September 9th, and lasted until some time during the early morning of September 10th. The rating table for San Pedro Creek is based on current-meter measurements for discharges of less than 200 sec-ft., and is extended for discharge for more than that rate by Kutter's formula. Above a 6-ft. stage, the lined section overflows, and the rating is subject to large errors.

C. Terrell Bartlett,* M. Am. Soc. C. E. (by letter).—The most important lesson brought out by both the San Antonio and Pueblo floods is that the experience of one or two generations is not sufficient to estimate safely the maximum flood dangers. The memory of the public in general is short, and in the many parts of the United States that have been settled only within the last 50 to 100 years, engineers, in too many instances, have based their judgment of flood discharges on the experience of a single generation or less, without thorough investigation of the traditions from even the preceding generation.

Where reliable records of flood discharges cover a period of less than 100 to 200 years, it appears that the most reliable estimates of maximum possible flood intensities can be based best on comparisons of possible rainfall intensities with the storms that have produced floods of known dimensions. In cases where such studies indicate flood magnitudes far in excess of the experience of one or two generations, and in cases where absolute safety is requisite, the engineer must have the courage to stand by his convictions in providing for the unprecedented. Had it not been for the old Spanish record of 1819, it is doubtful whether any engineer would have estimated a flood discharge more than double the greatest otherwise known at San Antonio.

Mr. Hazen's faith in the law of frequencies is amply supported by the record of that San Antonio flood. Except for this fragmentary Spanish record of more than a century ago, the 1921 flood might well have been regarded an anomaly, upsetting all theories of probability. However, it is found, as stated by Mr. Sherman, that the estimated flood of 100-year frequency, as determined from rain-storm intensities, is closely substantiated by two floods 102 years apart.

Another clue to possible flood stages is in the geological structure of a stream valley. Where there is a well-defined flood-plain structure, an estimated maximum flood discharge that does not correspond with a flooding of such

* Cons. Engr. (Bartlett and Ranney, Inc.), San Antonio, Tex.

valley, may well be questioned. An instance of this kind is in that part of the Rio Grande Valley in Southern New Mexico, known as the Mesilla Valley, where there has been no overflow of the entire flood-plain in historic times, but where Indian tradition tells of destructive floods. Fortunately, this valley is now protected by the Elephant Butte Dam.

With reference to the question of the discharge rates of the various streams at San Antonio, raised by Col. Jadwin, all the estimates are based on measurements of slopes and cross-sectional areas made after the flood, and no great accuracy can be expected. On reviewing the various estimates of flood discharge, the writer is of opinion that the values given in Table 3, may be finally recorded as the most probable crest discharge rates of the floods of September, 1921.

TABLE 3.

Stream.	Area of water-shed, in square miles.	Crest Discharge, in Second-Feet.	
		Total.	Per square mile.
Olmos	32.4	31 000	956
San Antonio	41	23 700	578
" "	45	16 000	356
Alazan-Martinez	16.9	28 000	1 656
Apache	22	19 000	864

The variation in discharge rates per square mile does not seem unreasonable, on considering the differences in slopes, topography, and rainfall and, in the case of the San Antonio River, the pondage in the city.

The timely discussion by Mr. Ellsworth is of great value in showing the degree of accuracy obtaining in the estimates of discharge made by the Geological Survey and in correcting a general statement of the writer, which might have been misunderstood.

As a resident of San Antonio and an observer of the 1921 flood there, the writer is impressed with the importance of the human or social aspects, as contrasted with purely economic considerations, in determining the extent and character of flood-prevention works. In cases where possible loss of life is a factor and where there may be many instances of ruinous individual losses, rather than a measured financial loss distributed on a whole community, the expenditures for preventing floods may justifiably exceed the amount computed by capitalizing the probable average annual flood losses.

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Paper No. 1486

ODORS AND THEIR TRAVEL HABITS*

By Louis L. Tribus,† M. Am. Soc. C. E.

With Discussion by Messrs. Paul Hansen, George C. Whipple, Stephen DeM. Gage, I. S. Osborn, Rudolph Hering, Olin H. Landreth, Andrew J. Provost, Jr., Robert Spurr Weston, Alexander Potter, Caleb Mills Saville, George A. Soper, and Louis L. Tribus.

Synopsis.

A famous educator on being asked to explain the term "synopsis" replied by saying that it was an "epitomized exegesis". It is difficult to present a synopsis of this paper, for it is more a rambling collection of notes about "smells", with some incidents illustrating their intensity, prevalence, persistence and movements, than a logical engineering presentation of a definite subject, with conclusions, as a suggested guide to follow. In fact, definiteness as to smells is impossible, even as to any standard of describing them, and the paper makes a point of that fact.

The tolerance of odors given off by essential industries, if due care is taken to reduce offense to the minimum, is considered, the effect of odors upon persons of different susceptibilities is discussed, and the fact that certain races accept as pleasing those odors which produce even disgust in others is noted. Some notes are given in regard to difference of behavior during daylight and night, particularly the curing effect of direct sunlight.

Observed distances of travel of odors are given in numerous instances, and various cases from litigation are reviewed, giving opinions of unnamed experts as to their views on distances to which odors travel, and the atmospheric conditions under which they become specifically obnoxious.

In general, the types of odors reviewed are those from garbage treatment and sewage disposal works, with some comments on the production of offensive gases in the preparation of food products, ore and oil refining, etc.

* Presented at the meeting of November 16th, 1921.

† Cons. Engr. (Tribus and Massa), New York City.

The writer's conclusions are that many engineers should give items from their own experiences, and thus provide a reservoir of information that might be helpful in settling various disagreements and even litigation; that chemists might devote more attention to counteractive work; and that some consideration might be given to amendments to public health laws to make possible the stopping of prospective nuisances without having to wait for their actual development before effective action can be taken.

The dweller in the country can generally count on fairly pure air, unrestricted light, good water, and, with a little intelligent care, suitable drainage and sanitary disposal of wastes. The city or townsman exchanges his freedom from drudgery for a dependence on united efforts to secure the necessities of community life, and encounters thereby many experiences in a lifetime where actual nuisance prevails, detrimental to comfort, if not perchance to health.

Odors (which, when offensive, are usually called "smells") are given off in the conversion of raw materials into food, oils, many metals, gas, and a countless list of articles for human use; and, again, through their passage back from such use either to their original or to transformed components.

Some of the objections to such conditions are sentimental rather than otherwise, but the result is the same—vigorous objection giving rise to talk, litigation, and sometimes actual violence.

The personal equation is a strong factor in considering questions of nuisances and their elimination, for there may well be a tolerance bred by acquaintance, or so overwhelming a need for the product, that humanity accepts the conditions without rebellion.

Only recently some of the older children and adults on Barren Island, Brooklyn, N. Y., were asked "What do you most desire?" The response was unequivocal, "Give us back the operation of the garbage plant." With all its atrocious features, it meant employment, money, and upkeep of their homes.

Garbage and Sewage Phases Considered.

This paper presents items chiefly from the garbage and sewage phases of the question—for they have come more directly into the writer's purview—and the traveling qualities of odors therefrom, rather than the methods of production.

A point of interest raised, about which there might be much discussion, is the advisability of making lawful the veto power by Health Commissions or Courts on projects that practically promise the creation of nuisance, rather than waiting for full construction and actual accomplishment.

As far as "smells" go, there is a wider field for study in counteraction than prevention, for the latter is costly and frequently litigious.

It may be venturesome for an engineer having only an elementary knowledge of the chemical and microscopic constituency of gases and their structure to attempt any characterization of odors (or smells) and their habits of travel. However, having been called at times to make use of nasal, visual, and sometimes stomachic susceptibility where questions of nuisance from odors had

been raised, the presentation of some notes may be pardoned, in the hope that discussion may bring together more data, crystallization of facts, and agreement as to procedure for the benefit of community welfare.

Mathematical Measurement Impossible.

To begin with, from a mathematical viewpoint the case is hopeless. One cannot say that a particular odor, given off at a specific heat, meeting a certain open-air temperature, at a special barometric pressure and known degree of humidity, with a wind blowing just so many feet per second over a level plain, will be clearly noticeable at a computable distance from its source.

Therefore, why should any solution of the case be sought? Simply because odors do travel, and opinions are desired which may result very tangibly in settling litigation, where questions of damages and injury to health are at issue.

From an article in *The Lancet* are extracted the following passages:

"If more studied the sense of smell might have been cultivated * * * and the study of odors would have reached a higher plane, though it appears to us the classification of smells must always suffer from the absence of the mathematical analysis capable in optics and acoustics. As it is, in our struggle to describe a smell, we are confined to using such terms as 'peculiar, persistent, pleasing, disagreeable, sour, bad, musty, sickly and so forth'. Now these are all adjectives which give us little enlightenment * * * as compared with color and sound and light mathematical laws. * * * Perhaps sometime smells may be also arranged according to some scale."

The foregoing is quite true; probably six simultaneous observers of six odors, describing them independently, would use not the same six adjectives but at least twelve or fifteen. The real point of importance, however, is the distance that smells travel and their actual effect on human beings.

State Laws Control Nuisances.

State laws usually empower boards of health to control nuisances, nuisances being generally described as "such conditions as offend humanity, either personally or through injury to property, or that may be considered as even detrimental to public health."

Decision is largely dependent on such individual cases as may be in point, and the enforcement of restrictions of noisome operations is dependent largely on the locality and the importance to the public of the operation itself. Some trades are essential, yet offensive; they may be tolerated in one place and debarred in another, yet with equal justice.

The second factor of importance brings in the human equation. Some individuals possess a keen sense of smell without effect other than objection to the disagreeable; some are violently affected; others are less keenly, although nervously, affected, and in the end seriously so. The symptoms in most cases are usually loss of appetite, headache, or even nausea.

Some races may not only tolerate but positively enjoy an odor that produces rank disgust in others. All know also that sometimes a whiff of an odor is offensive, while a well saturated atmosphere brings toleration; at other

times, and more usually, the reverse follows, consequently, no definite rule could be of universal application.

The Seal and Cod Liver Oil Works of Newfoundland, to the casual visitor, are offensive in the extreme, yet those living near-by become accustomed to the odors, which are typically rancid and more than suggest putrefaction. Fortunately, they do not carry far from the plants, or else they become so quickly mixed with the strong ocean air as soon to pass from recognition. They are essential local industries, and hence they are tolerated even if perchance disliked.

Control by Anticipation.

As yet the law does not permit control by anticipation; actual nuisance must be first produced; then, however, action may be swift and effective. Would it not be wiser, under well safeguarded acts, to permit legal review by boards of health before consent is granted to invest capital, where noisome odors might be anticipated? Of course, there are injunction proceedings open to taxpayers, but if State or municipal consent was first required, much litigation might be saved in later complications.

However, legal consents or litigation throw no light on odors traveling, the present point of special interest.

Special Cases Cited.

The writer recalls two cases during his boyhood, one a tomato catsup factory which kept in the open yard several tanks of pulp, just how far the pulp was advanced in putrefaction and how much was bottled for sale not being of present importance, and the fact is that, although very offensive within 100 ft., the odor could rarely be noted farther away. Operations were closed by order of the health authorities, but on grounds of unsuitable materials rather than general nuisance.

The other case concerned a cream of tartar establishment, where apparently due care was observed in manufacture, yet the vile odors traveled with almost increasingly disagreeable qualities to a distance of at least a mile under almost any atmospheric condition. In this case, a shut-down was soon effected on the score of nuisance, though not necessarily directly injurious to public health. The plant was located at the edge of a populous community, and could not be considered an essential industry.

The garbage reduction works on Barren Island (within the Borough of Brooklyn, City of New York) were for years synonymous with stench. Fumes from the retorts and tankage driers passed off in part through the chimneys from which, according to wind and humidity, they traveled 5 or 6 miles, or more, before diffusion and chemical break-up relieved the offense.

These odors, while disagreeable, were not particularly nauseating to most people, and for many years legal actions to end them were not effective. Recently, however, a permanent injunction has been granted. Close to the plant, although intensified, the odors were scarcely more offensive than at considerable distances.

Experience at New Works on Lakes Island.

The great reduction plant, built in accordance with many advanced ideas, and located on Lakes Island to replace the Barren Island works, for the several months of operation prior to its being closed as a nuisance, gave off gases that persisted in nauseating quality and strength to points 8 and 9 miles away. Rather curiously, although twenty-four hour operations were maintained, these peripatetic smells became more offensive after sundown. Evidently, the sun's rays possessed a deodorizing power which was joyfully welcomed by the burdened population through those months of torment.

Years ago, it was discovered, practically by accident, that sunlight streaming into tanks holding tried out seal oil, while going through the curing process, killed in large measure the exceedingly vile odors of decomposition, and thus made life endurable for the workmen.

The odor problems of many cities, aside from offensive trades, are those from garbage works and sewage disposal systems. A great many reams of paper have been covered with lawyers' briefs and thousands of witnesses have been examined in trying to solve the difficulties due to noisome smells; individuals attacking communities and private corporations, and communities attacking each other.

New York Case Against Bayonne, N. J.

A long drawn-out action was that of New York State *vs.* New Jersey, trying to enjoin the passing of corrosive gases and offensive odors from certain great manufacturing plants in Bayonne, N. J. Although no summary closure was effected, a very great amelioration was secured. A rather noteworthy step was taken by some of the defendants in gathering the worst gases into two great chimneys, and passing them into the air at a height of 360 ft. or more above ground. Although the eye can see the output drifting for several miles, diffusion and perhaps transformation largely occurs before the odorous gases descend to ground level again, unless a strong wind blows them earthward to produce their old-time results of throat and nose irritation (fumes from copper smelting predominate). Such fumes quickly kill vegetation.

Another bitterly resented crop of fumes came from great manufacturing establishments, again along the New Jersey shore, at Edgewater. In spite of the mile of Hudson River water between the shores of what is virtually a great canyon formed by the high banks, residents along Riverside Drive, New York City, and other neighboring streets, were driven almost frantic at times by the peculiar and irritating gases given off in the preparation of certain foods, chemical, and other commercial products.

Here, again, litigation and pressure have forced great changes for the better, but the point of present importance is not the peculiarities of litigation or the fact that usually such pressure brings some relief, but rather that odors do travel. Of still greater importance is the fact that such odors can usually be anticipated and controlled.

Gas Manufacture Now Inoffensive.

The earlier manufacture of illuminating gas was attended by odors from the wastage of the first output of the retorts and from the usually open tanks

of coal-tar and other by-products. Unless disseminated by strong winds these odors did not, as a rule, make themselves known at great distances, rarely more than ½ mile, but much litigation ensued nevertheless, particularly in Great Britain. At present, with the use of oil so largely replacing coal in illuminating gas manufacture, there is less of the waste from retorts, and the by-products, having commercial value, are conserved at once for sale. As a consequence, gas-making has nearly left the ranks of offensive industrialism, barring occasional accidents or carelessness.

In considering odors which give offense to communities, those from fuel consumption are largely ignored, for when serious in nature, it is more because of mixture with unconsumed carbon in the smoke, rather than the irritating or noisome constituents themselves; furthermore, dwellers in cities are so thoroughly saturated with ordinary coal gases as to have largely passed the stage of noticing them, unless they are present in great excess.

Soot as a Nuisance.

The soot elements have been productive of many a strenuous fight, and will still continue to do so until that day comes when, with power plants at the coal mines, electricity will be developed to do much more of its cleanly work in the cities, with the tremendous saving in labor, space, wastes, etc. The imagination could well run riot over the practical advantages of electric power, and with only a trifling betterment in mechanical conditions, the ultimate economies also. Of course, water power converted to electricity will produce the same result.

A public commission in Idaho, not long ago, issued its opinion, after obtaining much testimony, that electricity cannot take the place of the direct use of coal for heating; another body in Ontario came to a somewhat similar conclusion. Positiveness, however, is dangerous and the "cannot" of to-day becomes the "accomplishment" of to-morrow.

Odors Carried Long Distances.

Long before land can be seen from the ship, the peculiar and rather pungent odors of tropical vegetation in the West Indies can be discerned, indicating their travel of 25 or 30 miles at least. The ocean waters themselves may be somewhat responsible, due to stream flow and land washings, but it is atmospheric carriage in largest measure.

That leads to another phase of the subject, where possible absorption of gases by water, with movement to new scenes of activity before final transformation or diffusion of obnoxious elements may tend to a later separation, again of gaseous products, and to the extent of causing a transplaced nuisance. This has been often noted in sewage cases, where volatile elements from treated sewage effluents seem to have been carried in actively flowing streams for considerable distances.

In such cases, however, unless atmospheric conditions are very favorable, these finally liberated gases do not usually travel far from the stream itself. Even in the case of the Lower Passaic River, Gowanus Creek, and formerly

the Chicago River, the well-known trio of unenviable waters charged to practical saturation with sewage, the odors were not noticeable usually for more than a few blocks away from their banks.

From a recent report of the Albany Sewage Disposal Plant, the following statement is quoted:

"There is practically no nuisance caused by odor from the plant, as it has never been discernible for a greater distance than 300 yd., and then only on muggy summer nights."

Odors from Garbage Treatment and Sewage Disposal.

Returning, however, to the fields in which many communities have a common interest in either providing against prospective nuisances or in curing them when created, namely, garbage transportation and treatment and sewage disposal, a brief word on processes may not be amiss.

In the treatment of garbage, aside from field spreading, three distinct methods are in vogue, each with its strong advocates, and each, of course, with its various modifications:

(*a*).—High temperature incineration (1 300 to 2 200° Fahr.) where, under proper operation, no offensive odors pass from the plant except that a slight "caramel" aroma may at times be noted in the fleecy, whitish smoke that comes from the chimney, but is scarcely objectionable except to the ultra-sensitive. We do not speak of esthetic objections, but rather of sensatory causes.

(*b*).—Reduction by cooking, according to one or another of several processes. To date, except from perhaps very small scale operations, odors are given off from all practicable plants, having quite distinctive characteristics of aroma, persistence in staying qualities, and decided diligence in travel. These have already been referred to in the cases of the Barren Island and Lakes Island plants, of different types; the same conditions prevail at Boston and Cleveland, and perhaps not quite so noticeably, yet actually, at Columbus and Chicago. In the latter case, the situation is such that other kinds of odor-producing establishments are near-by, so that any one is less conspicuous.

(*c*).—Transformation by hog feeding. This became during the World War a popular and intensive process and a profitable one, but less so now that the price of pork has dropped back nearly to old-times rates. Properly managed, it may not contribute largely to community smells, but in olden days the reputation of hog farms was not enviable, their sweetish and swill odors being distinctly noticeable at considerable distances.

The Lakes Island Case in New York.

The Lakes Island (New York City) reduction case, being quite recent and very hotly contested, might be reviewed at some length, to indicate the lines of thought developed and opinions brought out. It progressed in three stages, as follows:

First.—The objection of residents of the Borough of Richmond, New York City, to having erected within its confines a plant planned to reduce

about 2 000 tons or more of garbage per day produced by three of the other Boroughs, entailing also the transportation of such garbage on scows closely paralleling for 5 miles the course of the Municipal Ferry which carries about 45 000 passengers daily, and passing along and within a few hundred feet of about 10 miles of Staten Island's water-front.

The State Commissioner of Health held a long series of hearings, calling to sit with him a Professor of Sanitary Engineering of Harvard University and a Deputy State Attorney General. The case was conducted by the District Attorney of Richmond County for the protestants; by an Assistant Corporation Counsel of New York City (defending the contract); and by counsel for the Metropolitan By-Products Company, the prospective erectors and operators of the plant. A great deal of testimony was taken, the hearings extending over many weeks, resulting in a report to the Governor that a nuisance would probably develop, but that until such nuisance actually occurred, the State law would not permit action. The Governor instructed the Commissioner of Health to keep close watch, and directed the District Attorney to report also to him at the first indication of trouble, so that he might take summary steps for abatement.

Second.—The residents of Richmond were not satisfied, even though having full confidence in executive intent, and, consequently, a Special Grand Jury took cognizance of the case and through the District Attorney and special witnesses, became advised of many facts, but again found that limitations of law prevented effective action. The construction of the plant had progressed, however, nearer to completion and early operations fully verified anticipation, that while perhaps it was possible to operate without offense, it was not practicable so to do. A presentment but no indictment followed.

Third.—The New York City Health Commissioner took a hand, operations on nearly full scale having been undertaken, and after hearing much testimony, expert and lay, very quickly decided that the nuisance was actual, intolerable, and unjustifiable, and forthwith forbade further operations. Combined with this official "order" came bankruptcy of the Company, and the residents were relieved of their troubles.

The odors were of nauseating quality and were seriously offensive at distances of at least eight miles from the plant. The discriminating nose could detect three characteristics in the stench: that of caramel or burning vegetable matter, semi-rancid decomposing swill (combined vegetable and animal elements), and of the solvent (a chemical midway between gasoline and kerosene).

These odors would settle in topographic and atmospheric pockets, and treated the public to many surprises as to their lasting qualities and intensity. The effect on different systems was quite diverse, producing severe headaches in many, nausea in others, lassitude in some, and generally violent wrath in all.

Varying Opinions of Experts.

In the two earlier proceedings the opinions of experts varied widely, several testifying for the Company. The City took the position that odors of rotting garbage, conveyed on open scows, if covered with tarpaulins, could

not escape and would not be noticeable more than about 100 ft. away, at worst. Others stated that from the plant built and operated as promised there could be no escape of gases to annoy even visitors to the property.

Theoretically, the last condition should have been true, except for the unloading of the partly decomposed materials. Practically, of course, no such operations would be carried on as they should be, and hence the conditions that actually developed should have been anticipated, in part at least.

There was much testimony offered as to the prospective and real qualities of the gases to be produced, and as to those actually emanating from the plant. This testimony tended, in weight of opinion, to support the claim of their being detrimental to health, even if not actually or violently poisonous.

There could be no divergence of unbiased opinion as to their injurious effect on real estate values, and as the loaded scows in service were not well covered, all theories as to their agreeable nature were "knocked endwise."

Details of Methods.

As to process, the following may be helpful to an understanding of the situation. The scows holding from 300 to 500 tons of garbage from 2 to many days old were towed from the Bronx, Manhattan, and Brooklyn to and alongside an open wharf at Lakes Island along the west coast of Staten Island, Borough of Richmond, City of New York. Grab buckets lifted the mixture to hoppers from which belt conveyors carried it to the double kettles holding 5 to 6 tons each. Live steam, introduced into the space between the inner and outer kettles, cooked and vaporized the constantly stirred mass. As the watery vapors passed off through a check-valved steam pipe system, solvent was pumped in to digest the grease. After 8 to 12 hours treatment, the dissolved grease was drawn off for settling and barreling. The vapors were supposed to be condensed for recovery of the surplus solvent, and the fairly dry tankage was conveyed to the storehouse, first being roughly screened, to take out such bones and rags as had escaped the pickers while the raw goods were on the first belt conveyors. Cans, glass, table silver, paper, and a multitude of articles were taken out by the human pickers in that stage of the work.

All processes where heat would develop odor were supposed to be closed, but carelessness, leaks, and failure in some features of the plant, permitted the escape of smells that, as a stench, brought catastrophe.

Monticello Sewerage System.

Another recent instance of litigation over odors has grown out of the operation of the Monticello sewerage system. About ten years ago, that village called on the writer's firm to modify, if necessary, and complete the system that had been well started, the designing and supervising engineer of which had died.

As completed, the disposal works occupied a somewhat swampy area of rather tight soil and consisted of three covered sedimentation tanks, four open stone contact beds, and two large open under-drained sand filtration

areas, the processes being successive, and operation alternating. The effluent from the under-drains, and from the sludge bed (used about once a year) passed into a brook which a few hundred feet away united with another stream that carried the village street drainage, and a smaller one that also first passed partly through the village.

Tannery sewage, after going through its own detention septic tanks, entered the sewerage system, but apparently did not materially interfere with bacterial action in the disposal works, although it added at times elements of color from waste dyes.

At a distance of ½ mile from the works, the combined streams crossed a main highway, near which was a summer boarding house. The owner brought suit for losses of income, charging that odors from the disposal works had driven away the boarders.

The normal effluent from the plant was clear, practically colorless, and almost odorless. The odor could scarcely be noted 50 ft. away, and along the flow line of the brook soon became so mingled with the swamp smell as to be indistinguishable. At times, however, some carelessness of operation or accident, or flood conditions, carried an overflow of liquid from the sand beds into the brook, this liquid being the result of two processes of bacterial break-down. Also, under certain atmospheric conditions, while the twice treated sewage was standing on the sand beds, slowly percolating to the under-drain, odors rose and sluggishly moved over the nearly level valley, then combining with the plaintiff's own cattle and hog yard smells and the natural swamp vapors, produced an unpleasant effect.

A great deal of testimony was taken in the many days given to the trial, tending to substantiate the claim that sewage odors did at times travel the ½ mile or more. Whether such fact could carry financial damages was solely within the province of the Court to say. It was clearly brought out and acknowledged that the plant itself was suitable, and properly installed.

The consensus of testimony was to the effect that atmospheric conditions as to humidity or dryness largely affected the lasting and persistent nature of the odors, and that light winds would carry them to considerable distances, while heavy winds would cause rapid diffusion and soon put them "out of business."

The Court rendered a rather unique decision, awarding $1 200 damages in the $25 000 claims, divided $400 payable by the village, $400 by unknown offenders, and $400 by the plaintiff himself as a contributor to the trouble. The village is also directed to enlarge its plant, so as to care more adequately for the maximum sewage load.

General Testimony on Odors.

Writers have in general expressed themselves rather conservatively, as a rule, when treating the subject of odors, for except when personal experience is available, little that is conclusive has been brought out.

One well known treatise on sewage states that "odors of aerobic treatment are less offensive than from anaerobic." In other words, open-air oxidation

gives off less putrid gases than closed bacterial methods. In another passage, the same treatise states "fresh sewage on trickling filters is not noticeable over 100 yards away * * *, from septic tanks some one-half mile." Another writer states:

"The odors which arise from decomposing sewage do not cause typhoid fever, or smallpox, or scarlet fever, or any other contagious disease. They may, and often do, reduce the comfort and happiness and, consequently, in a sense, the health of the people subject to them, but it cannot be maintained that they cause serious sickness."

From the testimony of experts in the Lakes Island case, the following brief sentences are quoted:

"Odors travel greatly under light air and high humidity."
"Garbage scow smells are noticeable from one-third to one-half mile."
"Odors travel 3 to 5 miles, increasing with quantity, * * * night travel is more offensive than day."
"Odors carry, say, three miles."
"Odors not so offensive with high winds."
"Barren Island odors are from the drying plant."
"Garbage odors will travel long distances."
"Could not smell plant (garbage) over 500 ft."
"Odors would not carry far."

Two of the Southern States a few years ago fought out very conclusively the principle that people are entitled to fairly pure air, as well as pure water; in that case, the fact of the travel of odors to an offensive degree and unreasonable distance was well established.

Recent Developments.

The phase of the subject considered in this paper is more that of odors which are offensive to people, than gases which cause damage to property. The principle is becoming more and more firmly established that upon the party giving the cause of annoyance devolves the duty of relieving it. In matters of sewage outflow into, and water supply from, the same stream there is a duty charged on both interests, first, to mitigate offense, and, second, to prevent danger; the right of drainage is held to be practically equal to the right to a pure water supply, so that treatment works are required to be operated by both parties.

At one time there was a great hue and cry about slaughter-houses in cities, and the offensive smells that came from them and passed liberally through the neighboring community. Many actions were brought to close them, but as a rule the Courts determined that the service was needed by the public, and Boards of Health were instructed to keep very sharp supervision to prevent needless annoyance.

Later, as a greater demand has developed for cleanliness, slaughter-houses have gradually almost ceased to be offensive. Economic questions also came into the case, for what had once been wasted and allowed to rot and smell, until sporadic house-cleaning was indulged in, became of value as commercial by-products. Blood once passing into sewers goes into feed and fertilizers; hair, hide, bones, horns, etc., all have market uses, and the offal in some

measure finds a sale; even considerable imports are brought to the United States of the entrails of animals killed in Japan and China.

To attempt to analyze an odor is hopeless, although it is certainly some form of emanation that has motive power of its own, but when sufficiently diffused is broken up into elements that cease to have original "personality", yet as long as traces last some characteristics persist.

The writer has visited many garbage treatment works and has noticed the clinging nature of the odors, which have often been retained by the clothing 12 or 15 hours after leaving the plant, being more noticeable from reduction than from incineration works. This is not indicative so much of traveling capacity, as of the simpler or more homogeneous form of the smell atom; it therefore does suggest a persistent character, hence, the ability to make itself known at great distances from the source.

Experiments have been made as to the toxic effect of garbage odors, but the results are not conclusive enough to warrant much discussion.

In the manufacture of linoleum and oil cloths, various oils are used, which, in changing to fixed forms, give off smells that permeate the air for sometimes several square miles around the plant. The preparation of linseed oil also produces smells that carry considerable distances. In neither of these cases, however, is there a condition that seems to develop unhealthful reactions in humanity.

From the foregoing and similar facts it may be asked, "Should and can odors be classified by any scale of descriptive terms? Can odors be analyzed to discover the atoms that make them agreeable or offensive, and thus find what other atoms may be needed and used to neutralize their influence?"

A homely instance is the cooking of cabbage and its filling a house with the characteristic and not altogether agreeable smell; a little soda in the water in which the cabbage is cooked practically neutralizes the odor without injurious effect. Parasites are grown to kill off other species detrimental to crops. Antidotes for many things that cause trouble are used.

There are, of course, well known germicides, but these are of greater service in arresting or counteracting putrefaction, than in transforming the gases already developed, unless to mix with them and by combination change their identity.

The work to be done may lie more in the field of the chemist, but the causes of many odors occur in the field of the engineer, so that from his knowledge and experimentation also there may come a solution for many a community's fears and discomfort, and prevention of the development of many an actual annoyance.

DISCUSSION

PAUL HANSEN,* M. AM. SOC. C. E. (by letter).—The author deserves great credit for preparing a paper on the elusive subject of odors. As he has pointed out, it is a subject which is so difficult to treat in a tangible and scientific manner that engineers have hesitated to make it the subject of discussion before technical societies. It would seem, however, that a fuller discussion of the subject, on the part of engineers and chemists, might lead to more standardized, definite, and scientific means for measuring the quality, intensity, and carrying powers of odors, than are in use at the present time.

As the author invites experience from other engineers, it might not be inappropriate to refer to a report on odors from an oil refinery, recently prepared by the writer in collaboration with F. C. Dugan, Assoc. M. Am. Soc. C. E., Chief Engineer of the State Department of Health of Kentucky, and G. C. Smith, Professor of Engineering Chemistry at the University of Cincinnati. This investigation seemed to call for some general remarks on the subject of odors, which are quoted essentially as follows:

"The effect, psychology, and significance of odors constitute a rather confusing subject especially to the lay mind and frequently to the legal mind. A primitive sense of odor was apparently designed by Nature to (1) distinguish inedible from edible substances; (2) as a means of recognizing things and creatures; and (3), in some instances, as a warning against danger. Generally speaking, the disagreeable odors were those that indicated inedible substances and dangers. Persons instinctively associate disagreeable odors with dangers to health. So strong are these instinctive feelings that it is difficult to get people to believe that their health and lives are not in danger when in the presence of strange or disagreeable odors.

"As a matter of fact it can be very clearly demonstrated that odors in and of themselves are not dangerous to health, although they may indicate the presence of substances which if ingested or inhaled in concentrated form may have physiological results.

"In addition to natural odors, man's ingenuity has introduced many industrial processes that have odors. Most of these industrial odors when first encountered are regarded as disagreeable in varying degrees, certainly they are nearly always objected to when introduced near places of abode, not so much because they are hurtful, as because they prevent the free enjoyment of the atmosphere as Nature made it. Where industrial odors are accepted as a necessary evil, as is apt to be the case in a greater or lesser degree in all large cities and industrial communities, people become inured to the odors and either do not notice them or, in some instances, actually develop a liking for them. Grain mills, woolen mills, cotton mills, dye works, soap factories, slaughter-houses, sugar refineries, shoe factories, paper mills, rendering plants, tanneries, etc., all have characteristic odors, often mild, but sometimes rather strong, to which whole communities become accustomed and accept without complaint. There are certain industries, however, that produce odors so strong and so unpleasant that they are rarely permitted in residence communities. Among these 'offensive industries' are rendering plants, slaughter-houses, certain chemical processes, glue works, etc.; but even some of these odors are tolerated and accepted as a matter of course and as unavoidable. Perhaps the most notable example of this is the notorious stockyards' odor that frequently pervades the whole South Side of the City of Chicago, in

* Pearse, Greeley, and Hansen, Chicago, Ill.

which are included many fine residence districts. Apropos of this, the so-called inquiring reporter of the *Chicago Tribune* one day made inquiry at random of ten people on the South Side of Chicago regarding their attitude toward the odors from the stockyards. Only one stated that the odors were disagreeable, nine were indifferent to the odors, and one admitted that to him the odors were actually agreeable. In short, it is difficult to decide when an odor may be regarded as a public nuisance and when it may not be so regarded; the most one can say is that an odor is a nuisance if the public is not used to it and does not want it in the neighborhood. The same odor may not be a nuisance if the public is used to it or is willing to tolerate it for economic reasons or otherwise.

"Odors from oil refineries seem to fall in a middle class. Though some very sensitive persons may be nauseated by oily odors, the average person is not apt to describe them as distinctly disagreeable in concentrations in which they would ordinarily be found even in close proximity to a refinery. Nevertheless, the same average person would be very much averse to having these odors in and about his residence, much from the same point of view as people nowadays avoid living on streets extensively traveled by automobiles, or regard the odor from recently oiled streets as objectionable in their homes.

"In a broad way, the principle seems to be fairly well established in practice (and, apparently, in law as well), that essential industries even though odor-producing must be accorded an existence when once established. Recently, there has grown a tendency to regulate in advance the location of the more objectionable odor-producing industries by means of zoning ordinances or other regulatory devices, which accord suitable space to all industries, yet protect the property holders and stabilize real estate value.

"Some industries produce corrosive, blighting, and even dangerous gases, but these effects must be considered as distinct from odors, because certain of these gases have little if any odor. Zinc-refining and acid-making industries fall within this class. An oil refinery, however, could hardly be classed as objectionable on these grounds, as the concentration of escaping vapors is never so great as to affect vegetation or injure the respiratory tract. This is evidenced by the general freedom of employees of oil refineries from afflictions of the respiratory organs. Refinery vapors may contain certain substances that have a corrosive effect on metals and which may affect certain kinds of paints. These effects are most noticeable when the oil is rich in sulphur compounds. We are unable at the present time to make any more precise statement regarding the nature of these substances, but certain observed effects on paint, especially on the office building at the plant, indicate the presence of small quantities of some substance or substances capable of injuring lead paint. The testimony indicates that at rare intervals the paint on houses in the city is affected somewhat by these gases. Probably, most of the objectionable compounds come from the tail-house where unfinished oils are exposed and also from the agitator where gases containing sulphur compounds may escape in considerable volume."

In connection with the investigation and the report from which this quotation is taken, a limited number of observations were made on the effect of wind, temperature, and barometric condition on the dissemination of odors, in which the records of the United States Weather Bureau were found to be of great assistance and value. Owing to the limitations under which the investigation was made, however, these observations were not sufficiently long continued or sufficiently well carried out to warrant giving them any space in this discussion. However, they would suggest the possibility that it may be practicable to obtain a classification of odors according to some generally recognized standards. It may also be possible to trace the dissemination of

odors with parallel observations on meteorological and other conditions, so that it may be feasible after a sufficiently large mass of data has been accumulated to predict, under any given set of conditions, whether or not a sewage plant or a so-called offensive industry is likely to become the subject of complaint.

In regard to the author's remarks on the feasibility of preventing odors rather than having them abated, the writer wishes to emphasize the value of zoning ordinances in this connection. Zoning ordinances are now authorized in a number of States, and they constitute a most excellent means whereby odor-producing industries can be located in advance so that they will not prove to be a discomfort or injurious to property values.

George C. Whipple,* M. Am. Soc. C. E.—The author has done well to bring to the attention of engineers this vexing question of odors and their travel habits. This question is inseparably connected with sewage disposal, stream pollution, garbage disposal, and many chemical industries. Odors that are offensive to human beings and travel over wide areas are public nuisances and come within the scope of police power, a power exercised in this case by State or local boards of health. There is hardly a question in public health administration which is more troublesome than this matter of odors. Several things contribute to make just decisions relating to odor nuisances difficult: The question of whether bad odors affect health; differences in people as to their sensibilities to odors, which are not only individual, but sometimes racial; physiological problems which are not well understood; the effects of temperature, moisture, and air movement, meteorological matters which are directly concerned with the dissemination of odors; engineering problems of odor control by methods of prevention or by air purification; the problem of comparing the cost of such protective measures with the damage inflicted, a matter of economics; and the question of the appropriateness of odors to certain places, a psychological question. These factors are often so mixed and tangled that it is no wonder that the legal and administrative phases of the subject are difficult. Perhaps, the solution of particular problems may be made easier if these different elements of the subject are considered separately under the following heads: (1) Physiology of odors; (2) relation between odors and health; (3) meteorological factors of odor travel; (4) governmental control of odors; and (5) prevention of odors.

The Physiology of Odors.—The speaker has recently had the pleasure of looking over the manuscript of a forthcoming book on "Smell, Taste, and Allied Senses in the Vertebrates", by Professor George H. Parker, of Harvard University, in which a chapter is devoted to the anatomy of the olfactory organs and another on the physiology of olfaction.

There has been some controversy among physiologists as to the mechanism by which the so-called sense of smell is exercised; but it is now recognized that two classes of substances produce nasal stimulation, namely, irritants, which act on the free ends of the trigeminal nerve, and true odors, which affect the terminal cells of the olfactory nerves. This is an important distinction. Some substances, like sulphurous acid, are irritants and have no

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true odor; others, like the oil of vanilla, have a true odor, but are not irritating; but some substances, like tobacco smoke, affect both sets of nerves, although this may be due to the mixed character of the smoke.

It appears to be well established that both odors and nasal irritations are caused by actual substances carried by the air into the nasal chamber. The quantity of these substances required to produce sensation is almost infinitely small. The ends of the nerve cells are not uniformly distributed over the nasal passages, and in ordinary breathing the air does not come in contact with them. Sniffing, however, brings the air into those parts where the nerves are located. Diffusion of the air in the nose may carry small amounts of the odoriferous or irritating substances to the nerve cells, and, once stimulated, involuntary sniffing may occur. The air may reach the nerve cells through the nose during inspiration, as in the ordinary process of smelling, or the odors may be carried to the cells from the mouth during respiration, as in tasting food. Many so-called tastes are really odors; holding the nose prevents respiration through it and thus prevents the taste (odor) of castor oil, for instance, from being noticed.

The physical state of the minute particles which produce irritation or odor does not appear to be known. Possibly they are electrically charged. It is certain that they affect the nerves most when accompanied by moisture in the air. "They do not emanate from their source and disperse in all directions as sound and light waves do." They result from evaporation or volatilization and are carried by the air as it moves, being spread through it by processes of mixing.

In the nose the olfactory membrane is covered by mucus into which the olfactory hairs project and beyond which they do not extend. The mucus apparently dissolves the odoriferous particles which thus come into intimate contact with the olfactory hairs.

TABLE 1.—Quantities of Substances Required for Minimum Stimulation.

Substance.	Thousandths of a milligramme per liter of air.
Camphor	5
Ether	1
Sulphureted hydrogen	0.5
Citral	0.5 to 0.1
Heliotropin	0.1 to 0.005
Cumarin	0.05 to 0.01
Vanillin	0.005 to 0.0005
Chlorphenol	0.004
Oil of rose	0.0005
Mercaptan (garlic odor)	0.00004
Artificial musk	0.00004

The quantities of substances required to stimulate the trigeminal and olfactory nerve cells are extraordinarily minute. Many years ago, when studying the odors produced by microscopic organisms in water, the speaker found that oil of peppermint could be recognized when diluted with 50 000 000 times its volume of water, and that the essential oils found in several of the

algæ could be detected in dilutions of from 5 000 000 to 25 000 000. Various devices have been used recently by physiologists in studying olfactory acuity. Some of these olfactometers are most ingenious, especially that of Allison and Katz.*

As a result of various tests, it has been found that certain odors can be recognized in the quantities given in Table 1. Different observers give somewhat different values, as might be expected.

Messrs. Allison and Katz† have given a table, part of which is reproduced herein as Table 2.

TABLE 2.—Concentrations of Milligrammes of Chemical per Liter of Air.

Chemical.	Intensity of Odor.				
	Detectable.	Faint.	Noticeable.	Strong.	Very strong.
Ethyl ether	5.833	10.167	14.933	17.667	60.600
Chloroform	3.300	6.800	12.733	28.833	46.666
Ethyl acetate	0.686	1.224	2.219	4.457	6.733
Ethyl mercaptan	0.046	0.088	0.186	0.357	0.501
Pyridine	0.032	0.146	0.301	2.265	5.710
Oil of peppermint	0.024	0.032	0.109	0.332	0.348
Iodoform	0.018	.....	.....	.....	.....
Methyl isothiocyanate	0.015	0.039	0.067	0.108	0.144
Butyric acid	0.009	0.021	0.066	0.329	0.580
Allyl isothiocyanate	0.008	0.012	0.024	0.030	0.201
Propye mercaptan	0 006	0.020	0.028	0.043	0.054
Amyl thioether	0.001	0.007	0.011	0.012	0.015
Artificial musk	0.00004	.....	.....	.....	.....

It would appear from these data that the olfactory nerves are more sensitive than those which give rise to nasal irritation, but it is probably unsafe to state this as a general truth.

In his studies of the odors of organisms in water, the speaker has observed that as odors decreased in intensity they changed in quality. This is probably due to the fact that few odors are pure, that is, due to a particular substance, most odors are mixtures. When these mixtures are diluted, the less sensitive constituents lose their effect first, while the more sensitive ones persist; the result being a change of odor. This is also true in the case of substances which produce both irritation and odor, the odor may persist after the irritation has ceased, or when increasing in intensity the odor is noticed first and this is followed by irritation. Tobacco smoke is an example. There may be mixtures of substances which act chemically on each other, so that the result is actually a different odor, and not merely a mixture of odors. On a more intense scale, it is well known that one odor may mask another and that a strong agreeable smell may be substituted for an unpleasant one. The little known subject of ratios of odors underlies the practice of "deodorization".

Temperature has an important influence on olfactory acuity, an influence partly physical and partly psychological. Heat increases the evaporation and

* "An Investigation of Stenches and Odors for Industrial Purposes", *Journal*, Industrial Eng. Chemists, Vol. XI (1919), p. 336.

† *Journal*, Industrial Eng. Chemists, Vol. XI (1919), p. 338.

volatilization of most odoriferous substances. Cold air drawn into the nose chills the mucous lining and perhaps retards the concentration of the particles in it. Conversely, cold fresh air makes the scent more keen. Cold water drawn into the nose greatly reduces the power of detecting odor.

Another important physiological phenomenon is that of fatigue and exhaustion of the olfactory organs. The continual smelling of an odor exhausts the power to recognize it. This may be a matter of a few minutes, or it may be longer. Every one knows the phenomenon of "getting used to an odor". Every one knows that breathing fresh air removes the last traces of odors and restores the power to recognize those which are very faint. An ocean voyage does this to a marked degree. On approaching the Japanese coast and while still several miles from it, the speaker will never forget how, after ten days on the Pacific, he enjoyed the "land smell", the smell of the "good brown earth".

The quality of an odor is usually designated by reference to the substance with which it is associated. It is different from taste, of which there are four clearly marked qualities: sweet, sour, salt, and bitter. Odor designations are numberless and constantly changing, but attempts have been made to classify them. Haller's classification has nine groups, as follows:

1.—Etherial odors: odors of fruits, beeswax, ethers; three subdivisions.
2.—Aromatic odors: odors of camphor, cloves, lavender, lemon, bitter almond; five subdivisions.
3.—Balsamic odors: odors of flowers, violet, vanilla, cumarin; three subdivisions.
4.—Ambrosial odors: odors of amber, musk; two subdivisions.
5.—Alliaceous odors: odors of hydrogen sulphide, hydrogen arsenide, chlorine; three subdivisions.
6.—Empyreumatic odors: odors of roast coffee, benzole; two subdivisions.
7.—Caprylic odors: odors of cheese, rancid fat; two subdivisions.
8.—Repulsive odors: odors of deadly nightshade, bedbug; two subdivisions.
9.—Nauseating odors: odors of carrion, feces; two subdivisions.

Another classification (Henning's) has six groups, as follows:

1.—Spicy odors, such as those of fennel, sassafras oil, anise, and cloves.
2.—Flowery odors, such as those of heliotrope, cumarin, and geranium oil.
3.—Fruity odors, such as those of oil of orange, citronella, oil of bergamot, and acetic ether.
4.—Resinous or balsamic odors, such as those of turpentine, Canada balsam, and eucalyptus oil.
5.—Burnt odors, such as those of tar and pyridin.
6.—Foul odors, such as those of carbon bisulphide and hydrogen sulphide.

All these groups are artificial and unsatisfactory. They have a psychological rather than a physiological basis.

Relation Between Odors and Health.—Do odors affect health? This question cannot be properly discussed without drawing a distinction between true odors and irritants and without a clear understanding of what is meant by "health".

In a physiological sense, meaning a condition of the body in which it operates normally, the effect of odors on health is ordinarily slight; often there is no effect. Yet intense odors may cause reflex actions of the digestive, muscular, and secretory systems. A vile odor may make one sick to his

stomach or cause him to close his nostrils with his fingers, or run away. Even bad odors, however, do not cause disease, as the term is ordinarily understood; they do not shorten life. The irritants may cause smarting sensations and respiratory discomfort and probably are more closely related to health than true odors. Offensive odors may cause people to refrain from breathing deeply, may cause loss of sleep, restlessness, loss of appetite, and general malaise. These are certainly matters which appertain to health, and to some people they are of serious import.

If, however, the word, health, is extended to include the psychological as well as the physiological, if it includes human comfort as well as bodily functions, then odors do have an important influence on health. In addition to what has been mentioned, odors have strong powers of suggestion and give pleasure, discomfort, or disgust, as the case may be. They may interfere with one's quiet and repose.

Psychologically, people recognize the appropriateness of certain odors to times and places. The smell of fried onions in the kitchen may cause no complaint, but the same smell in the living room or sleeping room is objectionable, at least until nasal fatigue makes one used to it. The combination of nasal fatigue and a sense of the appropriateness of odors makes working under various conditions bearable, in a sewer, a slaughter-house, a garbage plant, a snuff factory, a candy store. Furthermore, such odors do not appear to affect permanently the health of the workers. The odors of a tannery in a residential district, of an oil refinery near a beach resort, of a garbage plant, sewage works, or rendering establishment in or around human habitations, and of a house drain in a living room, are regarded as inappropriate and are quite certain to cause complaint. Psychologically, also, it makes a difference whether the odors are local. In a village where all the people get their living from a pulp mill, no one seriously objects to the pulp-mill odors, but let a pulp mill move into an established community, and the residents not connected with the establishment will regard the odors as a nuisance. Some odors also are regarded as a necessary evil, such as the smell of manure around the farm, and the smell of fertilizer on the garden; while the same odor suddenly appearing from an unknown source causes complaint. People have become accustomed to the smell of coal-burning furnaces; they have not yet become accustomed to the smell of oil-burners.

People object to certain odors because of their suggestiveness, and this raises the question as to whether bad odors may be taken as an index of unhealthful conditions. There seems to be only one answer to this, namely, that an odor is a useful, but not always a safe, guide. The sense of smell in animals is utilized largely in their search for food. In human beings, it is used in the selection of food. Human beings rebel at the odors associated with putrefaction, a matter of instinct which warns against eating unsafe food. Odors of decomposition in drinking water are also warnings which should be heeded. The odors of the algæ, however, are objected to not because they suggest that the water is unsafe, but because they are out of place. Where food and water are concerned, the warnings of bad odors should be heeded, but the absence of bad odors is not a proof of safety. Odors as guides

to unhealthful atmospheric conditions are much less trustworthy. The extremely minute particles of odor-producing substances will travel through the air to far greater distances than bacteria, which, although tiny in themselves, are vastly heavier than the odor particles. It has been proved that the odor of a sewer or house-drain in the air of a room is practically no indication of the presence of bacteria from the sewer or the drain. It has been stated that such odors may be accompanied by minute quantities of chemical substances, which in larger doses are dangerous to the human system, but it has not been proved that these minute quantities are dangerous to health, although on this point Science has not yet spoken the final word.

The odor of coal-gas in a room has come to be regarded as an index of danger, the real danger being the odorless carbon monoxide; but water-gas, which contains much more carbon monoxide than coal-gas, has much less odor, and, for this reason, gas poisonings are more frequent now than before water-gas was largely substituted for coal-gas.

In spite of its shortcomings as an index of danger, in spite of its limited physiological bearings and its erratic psychological relations, the sense of smell is one which cannot be ignored, and human beings continue to insist that health and comfort be kept together and not separated artificially, as for the purpose of scientific discussion. The "health and comfort of the people" is an expression which not only has a great human urge back of it, but which has been found to stand the test of the Courts. In the speaker's opinion, the word, "health", used as a basis of the exercise of police power, should be given its broader and not its narrower meaning. In this connection, it is well to remember the following words taken from the first public utterance of the first State board of health in the United States, that of Massachusetts, in 1869:

"We believe that all citizens have an inherent right to the enjoyment of pure and uncontaminated air and water and soil; that this right should be regarded as belonging to the whole community; and that no one should be allowed to trespass upon it by his carelessness or his avarice or even by his ignorance."

Meteorological Factors.—Inasmuch as odors and nasal irritations are caused by actual particles (it matters little whether they are regarded as gaseous, liquid, colloidal, or solid), transported by the air from their source to persons affected, a study of odor travel is very largely a study of meteorology, especially air movement.

First, moisture in the air intensifies odors. It provides a solvent for certain substances which otherwise might not be given off at the source, and it more readily conveys the odoriferous substances to the nasal mucus. A dry air tends to harden the surface of the mucous lining and protect the olfactory hairs. A warm air also intensifies odor, as a higher temperature hastens evaporation and volatilization as well as the sensitiveness of the nerve endings. A low barometer also hastens evaporation. Rain and snow act as natural air washers and purify the air. Ozone is said to have an oxidizing effect, but little is known of this. It may be that there are also electrical conditions which have an influence on the almost molecular particles which produce odor. The greatest factor, however, is air movement, a factor which in itself is

dependent on temperature, barometric pressure, humidity, and other physical conditions.

Of the air movements, first consider those gentle movements, unmeasurable by the most delicate meter, due largely to temperature difference. They can be observed by movements of smoke and dust particles and by cooling effects on the skin. It is these almost insensible convection currents which spread the odor particles from their source for short distances in all directions. Very few odors are strong enough to be carried far from their source in this manner. Next, in order, are the gentle variable breezes which waft the odors to distances of a few hundred yards. They are usually intermittent, and not always constant as to direction, but they cover a wide angle. At a distance one gets the odor now and then. Finally, there are the winds, stronger and more constant, which carry the odors in a definite direction for long distances. The angle in this case is narrow. This pollution of the air by odors is not unlike the pollution of the water of a lake by sewage. The pollution may spread out slowly like a fan or, more accurately, perhaps, like a cone, or it may be carried by a strong current in a long narrow streak.

The speaker is not sure but what the author is right when he says that odors cannot be subjected to mathematical analysis. May it not be possible to divide odors into groups of different magnitude, define them in terms involving the velocity of the wind, the angle of the sector, or cone of pollution, and the distance from the source? The angle of the sector is a function of the veering of the wind, that is, the deflections of the wind from parallel lines, due to all kinds of minor local causes, which have a greater influence when the wind velocity is low than when it is high, as the higher winds are controlled by larger and more general causes. No classification should be attempted, however, until many careful observations have been made.

Topography, of course, is an important element in odor travel. Air travels through valleys, "draws" through breaks in hills, and is obstructed in its flow by forests and buildings. In any mathematical formula, these factors would enter as a coefficient which would vary with local conditions.

The elevation of the odor source has an important influence on the travel of the odor. Odors escaping from tall chimneys may be unnoticed on the ground near the base of the chimney, but may be very noticeable a mile away. On certain days, when the sun is shining on the ground, the air in the lower strata becomes heated and tends to rise; consequently, the odors become lost in the upper air. At night, the conditions may be reversed; the odors lie lower. The water surface of a lake or harbor influences the vertical movements of air in ways that are well known. The importance of vertical currents is more appreciated since the aeroplane has come into use. The humidity conditions at night are also more favorable for odor travel. The effect of damp, muggy days on the intensity of the odor nuisance is too well known to need further mention. Muggy conditions not infrequently accompany a lower barometer, a factor which also tends to facilitate odor production.

Governmental Control of Noisome Odors.—According to Judge Jeremiah Smith, late of the Harvard Law School, a nuisance is whatever the Court decides is a nuisance. Attempts have been made to define a public nuisance,

but in the last analysis the decision goes back to the opinion of "twelve good men and true" in a particular case. As far as the law is concerned, an odor nuisance, if action is taken by the Government under police power, must be proved to affect the "public health, safety, morals, or the like". The modern tendency is to extend the police power to cover public welfare and to let the phrase "and the like", cover public comfort, quiet, and repose. Certainly, it will be admitted that bad odors and irritating fumes are a public discomfort. In such matters as these, the opinion of the layman is often sounder than that of the expert.

American cities are growing, and as they become larger it is right that the police power should be extended and applied more severely. A slight odor affecting a large population may be as much of a nuisance as a very bad odor affecting a small population; for in the large population there is a greater chance that more persons susceptible to the slight odor may be present.

Bad odors which are spread over large residential areas, may depreciate property values, but the speaker fails to understand how this can be properly regarded as coming within the scope of the police power. In board of health hearings, however, this argument is commonly introduced. Claims for damages to property are for the Civil Courts to adjust. Lawyers know, however, that police power action by a health board is a help in a suit for damages, and not infrequently this is the motive back of nuisance complaints. One of the greatest difficulties in the governmental control of nuisances is to know whether the complaints are really sound.

Another serious difficulty in such control of objectionable odors from industrial works is that the nature and extent of the odor and the certainty of its being a nuisance cannot legally be determined in advance. In the Staten Island garbage case, referred to by Mr. Tribus, the Governor of the State of New York decided that the State's power of abatement could not be exercised until the works had been completed and had been found actually to be a nuisance, in spite of the fact that an investigation by the State Health Commissioner had shown that a nuisance would probably occur. This brings up the necessity of the exercise of greater care by local authorities in granting building permits for industrial plants. Local governments are usually glad to have a new industry established and readily grant a permit for building, often without bringing the matter to the attention of either the local or the State board of health. In the case of chemical manufacturing, the permitted establishment may create an odor which causes widespread and justified complaint. Then comes the difficult decision. Shall an established industry be closed? Shall the economic advantage to the community be tacitly considered in the exercise of police power? Shall economic advantage be considered as offsetting public discomfort? Or, if the case is one in which the odors can be prevented, how far shall the manufacturer be made to correct them? And if the industry is a new one, how long shall suffering citizens be made to wait while new devices for odor reductions are being tried? The lesson taught by many unpleasant experiences of this kind appears to be that it is wise to exercise greater caution in the location of industrial works which involve chemical processes, and to limit them to certain districts or zones.

The speaker was a member of the New York Commission on Building Districts and Restrictions, and his studies at that time led him to believe very strongly in the zoning idea. Zoning alone, however, is not enough. The Staten Island garbage plant was located in an established industrial district, but the trouble was that the odors "would not stay zoned". One way of avoiding the difficulty would be to require local authorities, before granting a building permit to a chemical industry, or any industry likely to cause public offense, to receive a letter of advice from the State department of health or other competent authority, such letter to be published and thus brought to the attention of the people before action is taken. Although the layman is as good a judge of what offends his nose as a sanitary expert, the expert is better able to judge in advance of the probability of a public nuisance being created. No such advisory action is needed in regard to the location of certain nuisances which have been adjudged by the legislature or the Courts to be nuisances *in esse*. Certain industries have well earned the title of "offensive trades", abattoirs, soap factories, rendering works, tanneries, and the like, as well as sewage works and garbage works. Already, the laws in some States require the location of such works to be approved by a sanitary authority. Just as health departments establish lists of diseases dangerous to the public health, so should they establish lists of industries which threaten the public health and comfort.

When a manufacturing plant, by disseminating irritating fumes or odors, becomes a public nuisance, the demand is usually made that it be shut down, heavy fines levied, or some other penalty inflicted. There are cases where such punishment is just and where the nuisance cannot be abated in any other way. In many instances, however, the elimination of odors is a difficult matter, and those who operate the plant do not know what to do; they do not feel sure that the expenditure of money for certain devices will accomplish the result. In other words, what they need is sound advice, not punishment. Unfortunately, the Engineering Profession has never developed specialists in odor prevention. This appears to be a field worthy of serious consideration. Combined with smoke and dust prevention and ventilation, it should furnish ample scope for a career for a young man who has a knowledge of chemical engineering, physics, meteorology, and sanitation. For many years State departments of health have been in a position to give advice in regard to the purification of water and the disposal of sewage and, in the speaker's judgment, the use of this advisory power has accomplished more than the sterner measures of the police power. The disposal of sewage, however, is practically a single problem, although, of course, it has its complications, and it is also a public problem. In the case of chemical manufacturing, the situation is different. The problems are very complex, the works are privately owned, and the processes are often secret. Hence, the consulting specialist is in a better position to handle the problem than the sanitary department of a State board of health.

Not infrequently, odors from industrial plants are due merely to bad management, carelessness, or sloppiness, and the correction of these faults

often leads to greater profits to the concern. The escaping fumes may be waste products, but they are sometimes valuable and worth saving.

New lines of manufacturing are constantly being taken up and new problems are continually arising. Thus, in Massachusetts, several oil-refining plants which represent investments running into several million dollars, have recently been built. Some of these plants are handling Mexican oil which contains much more sulphur than the common American products. The refining processes have given rise to odors and irritating fumes of an offensive character, which, in one notable instance, spread over a wide area and caused a serious public nuisance. Recognizing the novelty of the problems to be solved and the importance of the industry to the community, an order requiring the works to close was stayed from time to time, in order that the effect of new devices might be ascertained. Finally, however, there appeared to be no other course, just to the people, than that of closing the works until certain advised alterations could be made.

Odor Prevention.—This leads naturally to the subjects of odor prevention, air purification, and air analysis, which are too detailed for the speaker to discuss. Various methods are now available. The sedimentation of dust, the electrical precipitation of dust, acid, fumes, etc., by the Cottrell system, the washing of air by drops and sprays of water, the burning of organic dust at high temperatures, the filtration of air through cloth, the ventilation of buildings, the removal of odors absorbed by floors, walls, and fabrics, by washing and by the use of a compressed air blast, the use of disinfectants to prevent or arrest decomposition of organic matter, the application of sunlight, the use of chemicals to alter the character of the odor-producing substances—these and other methods, used singly or in combination, as the situation may demand, furnish means by which almost any industry may be made amenable to reasonable standards of health and decency.

Stephen DeM. Gage,* Esq.—Both the author and Professor Whipple have pointed out that much trouble might be avoided if some method was devised for locating odor-producing industries where they would be least offensive. The situation which exists to-day in certain parts of Providence, R. I., and the adjoining communities, is the direct result of such lack of foresight.

About ten years ago, the Chamber of Commerce and other public bodies endeavored to increase the importance of the Port of Providence, and a number of the large oil companies were induced to establish extensive receiving and distributing stations at the head of Narragansett Bay. To-day, Providence is one of the largest oil ports in New England. In connection with the operation of these oil stations, two companies established plants for the separation of crude oil into its component parts by distillation, and at one of them practically all the processes used in the refining of oil products are carried out. These two plants are about 3 miles apart, on opposite sides of the Providence River, and within about 4.5 miles from the center of the city.

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Unless plants of this kind are completely equipped with special devices to control odors and are carefully operated, offensive nuisances are likely to arise from odors which at times may travel considerable distances. Shortly after these plants began operation, complaints of offensive odors in adjacent residential districts were received, which increased both in number and in vehemence, until the Legislature directed the State Board of Health to investigate and report on the subject. During the progress of this investigation much was learned about odors of this kind and about their travel habits.

The odors from oil-distilling and refining plants are probably of a mixed character, partly gaseous and partly colloidal. They are variously described as "burnt oil", "burnt rubber", or "sweetish burnt rubber." Observers who have had laboratory experience and chemists generally use the latter term as most definitely expressing their impressions. The character of these odors varies considerably with the dilution. Near the source when they are intense, they have the characteristic odor of rotten eggs, or sulphide odors. As the distance from the origin increases, and they become more dilute, they take on the odor of burning rubber, with a sweetish or slightly burnt sugar flavor. With more dilution the rubber constituent disappears, and the odor may best be described as that noted when a baker's oven full of well browned bread is opened. When still more dilute, these odors are like that of slightly scorching paper. At first, these changes in the character of odors from the same source led to considerable confusion. In one instance, the speaker located a not unpleasant odor on the leeside of a large bakery, which was attributed to that source, and was surprised to find that this odor was much stronger on the windward side of the supposed source; and it became decidedly offensive as it was traced to its origin.

The reason for this change of character in odors is undetermined. It may be that odors are a blend and that one component is diluted out sooner than others. It is well known, however, that some apparently simple odors change in character with dilution. The characteristic odor of kerosene is recognized easily by almost every one, but when greatly diluted it is more likely to be described as some kind of perfume.

It may be of interest to mention that during the investigation mentioned considerable care was taken to obtain comparable observations from different observers. The acuteness of the olfactory sense varies widely in different people and in selecting observers for this work, men were chosen whose sense of smell was reasonably alike. These men were then taught to record odors of similar intensity in a similar manner. The scale was the same as for recording odors in water analysis, "very faint", "faint", "distinct", "strong", and "very strong", the distinct odor being one which would be readily detected by almost every one and which would be disagreeable to most people.

From observations, the speaker believes that the travel distance of odors of this character is mainly a function of two factors: the amount of odor produced at the source, and the velocity of the wind. There are other factors, however, which apparently enter into the problem. Weather conditions affect the travel habits of odors to a certain extent. The theory that sunlight may have a deodorizing effect may account for the fact that odors of this kind

are more troublesome at night or on days when there is a fog or heavy mist, than on clear bright days. As instances of odor travel, the following observations might be recorded:

1.—Night, weather clear, wind from 3 to 4 miles per hour, travel distance about 1.5 miles, area affected about 3 sq. miles.

2.—Day, rain, wind about 4 miles per hour, travel distance about 4 miles. In this instance, the odor was carried across the water about 2 miles before affecting a residential area of about 2 sq. miles.

3.—Night, cloudy with light showers, wind from 4 to 5 miles per hour, travel distance about 2.25 miles, area affected about 5.25 sq. miles.

4.—Night, cloudy, wind about 5 miles per hour, travel distance about 4.5 miles, area affected nearly 10 sq. miles.

5.—Night, weather clear, wind about 9 miles per hour, travel distance about 1.5 miles, area affected approximately 3 sq. miles.

6.—Night, weather clear, wind about 9 miles per hour, travel distance about 2.25 miles, area affected about 5 sq. miles.

One other interesting observation may be mentioned in which the direction of the wind was such that strong odors from two sources mingled and fortified each other. The characteristic odor was noted at a travel distance of 2.5 miles from the nearest source and about 6 miles from the more distant source. At this time a residential area of about 11 sq. miles was affected, in addition to an undetermined area over the water. This odor occurred at night, in clear weather, and with a wind velocity of about 5 miles per hour.

It should be noted that the wind velocity is that recorded at the Government Meteorological Station, 2 to 6 miles from the areas in which the observations were made. These figures are the only ones available, but the possibility that the wind velocity might be different in the two localities must be taken into consideration in any interpretation of the observations noted.

The author has pointed out that there are many apparent peculiarities about the travel of odors. With little or no wind, they do not appear to travel far from their source, and if the wind is strong, they are apparently dissipated and not troublesome at a great distance. In some instances, they may travel high, although at other times, with apparently similar conditions, they will travel low. There are a number of observations where these odors, quite offensive a mile or more away, could not be noted near the point where they originated. In one instance, an odor was traced from hilltop to hilltop for a considerable distance, but could not be detected in the valleys. Other cases have been observed of an odor settling in air pockets and remaining an offense long after it was discharged from its source. This is most likely to occur on damp, foggy nights with little wind. It has been also frequently observed that these odors will be strong on certain streets and perhaps cannot be detected in parallel streets on either side. Variations in odor intensity in near-by localities can be attributed only to differences in air currents. Such variation may be responsible for a wide difference of opinion among the residents of certain sections as to whether a certain industry is offensive or non-offensive, and such differences must

often be proved and reconciled before relief from offensive conditions can be obtained.

Another phase of the oil problem becoming prominent, which may become of greater and more far-reaching importance than odors from oil-distilling and refining plants, is the probable effect of the substitution of oil for coal in power and heating plants. There is no question that oil fuel is easier to handle, easier to store, and more economical than coal. The bulk of fuel oil used in New England cities is from Mexican crudes which carry about 4% of sulphur. The fuel oil contains more than 4% of sulphur, as much is left behind when the naphtha and gasoline are removed. If the combustion of this fuel is complete, the sulphur is converted into sulphur dioxide, an irritating gas which is discharged into the air. If the combustion is not complete, which is frequently the case, a part of the sulphur passes off as hydrogen sulphide or other sulphide gases, together with some partly burned oil, all of which are extremely offensive. Many oil-burners have been constructed in fire-boxes designed for soft coal and it is practically impossible to obtain complete combustion. Furthermore, some of the oil-burners now on the market are defective and cannot be operated to the best advantage.

For purposes of illustration, assume a community using 1 000 000 bbl. of oil fuel per year. This is not unreasonable, and the speaker knows of a single power plant now using this quantity. Only 4% of sulphur would amount to about 30 000 lb. of sulphur burned per day. With complete combustion, this would be equivalent to 60 000 lb. of sulphur dioxide gas discharged from the stacks every day. The speaker does not know that this sulphur dioxide when diluted with the air would directly affect the health of the people; he does believe, however, that if the oil-burning plants were concentrated and near enough to the residential section, it would cause more or less serious irritation to the nose, throat, and lungs of a certain portion of the population, and perhaps render these people more susceptible to other diseases. The speaker also believes that under prevailing conditions at many oil-burning plants to-day, the odors of incompletely burned oil and sulphides would be so frequent that this community would not be counted a desirable place in which to live. This is the problem to be solved in the near future. This problem and all odor problems can be solved, but the engineer and the chemist must first build a solid foundation of scientific facts before anything definite can be accomplished.

I. S. Osborn,* M. Am. Soc. C. E.—The subject presented by the author is a timely one, to which practically no attention, study, or research has been given. Had a small part of the cost of litigation in connection with suits brought to eliminate nuisance from odors been expended in study and research, there would have been accomplished far more toward the elimination of odors than has been achieved by Court decisions.

Since the speaker has had more or less intimate contact with all the plants mentioned by the author, the points raised in his paper relative to garbage disposal works and the odors produced are of especial interest.

* Cons. Engr., Cleveland, Ohio.

In dealing with the problem of garbage reduction, one must necessarily consider somewhat the size to which this industry has grown in the United States. In an investigation for the United States Food Administration in 1918, the speaker found that 29 of the larger cities disposed of their garbage by the reduction method, and produced annually 72 000 000 lb. of grease and 150 000 tons of fertilizer tankage, valued at $11 000 000. These figures demonstrate that the industry deserves considerable attention. The nature of the business, as well as the attitude of the men or the companies engaged in the operation of disposal plants, has not given much incentive to develop the industry along scientific lines, with the result that it has grown to this size without much assistance from the Engineering or Chemical Professions.

The plants have not been perfect either for the making of all possible recoveries or the elimination of nuisance from odors. However, the industry should not be condemned, but should have the efforts of engineers and chemists toward development whereby it can be operated without nuisance and without the objections due to past performance.

In this discussion of garbage reduction plants the odors considered are those which will travel or give rise to complaint in surrounding communities.

The reduction methods for garbage disposal vary in detail, but the majority of plants use the digester and dryer systems, in which the garbage is cooked in digesters, with live steam, after which it is pressed and dried.

In plants of this type the sources of odors are from the raw garbage, exposure of material during process, finished products, leaks in apparatus, vents from digesters, and gases from direct heat dryers. The greatest source of objectionable odors are the gases vented from digesters, or the gases from the dryers. The vent gases are the most penetrating and objectionable, but are more easily controlled and treated than the much larger volume of sweetish or caramel odors from the dryers. The odors from the raw garbage, the plant, and the finished products can be controlled and, with proper design and operation, should not give cause for nuisance.

In 1915, the speaker made an investigation and report to the Committee on Street Cleaning, Board of Estimate and Apportionment, New York City, on the elimination of odors from the Barren Island Disposal Plants. At the time of this investigation practically no data on this particular subject were available. This investigation covered the three plants operating at that time on Barren Island, namely, the plants of The Sanitary Utilization Company, which were then used for the disposal of garbage under contract with New York City; the plant of the Thomas F. White Company, which was operated for the production of tankage by drying the digested garbage; and the plant of The Products Manufacturing Company, which, under contract with the City of New York, disposed of dead animals and meat offal, and also rendered garbage collected from hotels and restaurants.

The assistance of The Central Testing Laboratory, maintained by the city, was obtained for analytical work in making these studies. The investigation which covered a period of about four months was for determining the necessary steps to remedy and eliminate the objectionable odors from these plants. Tests were made to determine the source of the odors, and the volume of the

gases carrying the odors, as well as the methods that might be adopted to deodorize the gases. The tests were made both in the field and in the laboratory, and, except the chemical analysis of gases which was made in the laboratory, the larger part of the work was done at the plants while they were in operation.

These tests involved the development of methods in regard to which no precedents and data were available. It was found that the apparatus secured for this work was not adaptable and it necessarily had to be remodeled, calibrated, and tested. Search was made for information on this subject as well as on kindred subjects. Although the work was not carried to the extent desired, the results of this investigation were, in many ways, satisfactory, many facts were established, and information of great value was gained regarding the design and operation of such plants.

Analyses of the gases given off from the vents of digesters before and after passing a barometric condenser showed that the carbon dioxide, which amounted to 45.2% before passing the condenser, was reduced to 6.2% after passing it, which indicates the efficiency of the condenser in absorbing condensible gases. The odors from digesters were found to be due primarily to the sulphur compounds and to a less extent to essential oils.

Tests to deodorize the vent gases entirely eliminated the odors by washing the gases in a solution containing small quantities of sodium and calcium hypochlorite (chloride of lime). It was found that 10 cu. ft. of gas could be deodorized in 150 cu. cm. of calcium hypochlorite solution containing 4 parts of available chlorine in 1 000. Gases treated by heating alone were not deodorized, but when heated to 1 100° Fahr., the sulphur compounds changed to sulphur dioxide which was soluble in water; so that by washing after heating, complete deodorization was obtained. The commercial application of the calcium hypochlorite solution in washing gases was difficult in practical operation, and the method of heating and then washing showed the greatest possibility of application.

Since this investigation, developments have been undertaken whereby gases carrying odors are treated with chlorine gas. The results have not yet been determined by analysis, although where the chlorine gas has been applied to digester vent gases, the characteristic odor is not noticeable at the point of discharge. It has not been determined whether the odor is disguised or masked by the chlorine gas or whether chemical action deodorizes the gases.

Where the direct heat dryers were used, the volume of gas given off was excessive. From 175 000 to 200 000 cu. ft. of gas per min. escaped from the dryers in the plant of the Sanitary Utilization Company. The cause of odors in dryer gases is due largely to the scorching or burning of the material and a sweetish and caramel odor is produced, due to the scorching of sugars. In making tests, dryer gases were completely deodorized by washing, the elimination of odors by scrubbing being in proportion to the degree they were washed. It is difficult in washing gases to eliminate the odor entirely, although it is possible. Heating the dryer gases to a temperature of 1 800° Fahr. eliminated the odor; they were also completely deodorized in calcium hypochlorite solution.

Experiments to ascertain whether the odor could be eliminated by the dilution of gases with air, showed that the odor was still distinct in 1 part of dryer gas with 400 parts of air; that it is characteristic of this gas to travel more or less in pockets and not diffuse readily with the air; and that it is not feasible to depend on the dilution of dryer gases to eliminate odors.

The elimination of odors from handling materials, as well as plant and room odors, is a problem to be considered in the design of plants. With suitable methods adopted for their control, and attention in operation, they should not give rise to objection. It is fundamental, however, in the operation of disposal plants, that the first object should be the elimination of the production of odor, thereby making it unnecessary to provide means for deodorizing.

The statement made by Mr. Tribus that, although it was possible to operate the Staten Island plant without offense, it was not practical, is not borne out by fact. The financial results obtained by the company would have been greater if the plant had been operated without offense.

The Staten Island plant uses the Cobwell process, which differs from the digester system. In the digester system, the garbage is sealed in tanks or digesters and live steam is admitted for cooking. The digesters must necessarily be vented to insure a supply of steam for raising the temperature of the material. After digestion, the material is pressed, and the solids are dried. In the Cobwell process, a reducer with a steam jacket is used, and the garbage in this reducer is submerged in a solvent for the grease, which acts also as a dehydrating agent. Water or steam vapors combined with solvent vapors leaving the sealed reducer pass through a vent pipe to the condenser, where they are practically all condensed. Tests showed that the vapors given off from the reducers, after passing the condenser, were odorless.

There is no question regarding the odors referred to by Mr. Tribus from the Staten Island plant. From the speaker's observation they were detected at a distance of from 3 to 4 miles. The odor was not the same as that from the plants on Barren Island, where the digester and dryer systems were used. In all probability, most of the odor given off was the result of the operation of the plant without proper sealing, which allowed the gases to escape directly to the atmosphere without passing through the vent line to the condenser. This is borne out by the results of investigation as well as from operation, and is the cause of the great loss in solvent which resulted from this practice.

The speaker was in charge of this plant during the first two months of operation, during which time it was found, not only possible, but practical, to operate without producing gases having odors. During this time, from daily inspection and study, the odors emanating from this plant were never noticed off the property on which it was located. The experience of the company in regard to loss of solvent demonstrated that it did not pay to permit odors to escape, for with the escape of any gas from the reducer, there was a corresponding loss of solvent.

The statement made by Mr. Tribus that slaughter-houses have gradually and almost ceased to be offensive is not fully borne out by investigations of this industry. In many cities, as in former years, they are either tolerated

as a necessary evil, or there is continual agitation and occasional litigation in regard to them.

Mr. Tribus states that "the real point of importance, however, is the distance that smells travel and their actual effect on human beings". Is it not of greater importance to determine the source and the necessity of the odor. In most cases, the source can be determined and the emission is unnecessary.

A survey of the past efforts and policies of municipalities with reference to this subject, reveals a lack of attention given to the problem. As long as municipalities expend no greater effort toward an engineering solution of the garbage disposal problem in the design of municipal plants, and continue the practice of awarding short-term contracts for the disposal of garbage, whereby the contractor must charge off his investment during this short term, attempt to pay dividends from the earnings under the contract, and, at the same time, meet competition as to price, just so long will the same results be obtained as at present, from a large number of plants.

This statement should not be taken to mean that all disposal works should be municipally operated, but each municipality should control not only the operation but the initial outlay in order not only to safeguard the city against objectionable methods, but from an economic standpoint as well. The continued unsanitary results, agitation, and litigation that have accrued are only natural, and so long as present practice is continued, so long will objections and litigations continue.

The statement by Mr. Tribus that the garbage reduction plant on Barren Island was "for years synonymous with stench", is true, but he is mistaken in that the plant was closed due to legal action. It was closed because the owners did not secure a renewal of their contract with the city. An injunction, however, was granted forbidding the operation of The Products Manufacturing Company's plant on Barren Island, where, under contract with the City of New York, dead animals were rendered. The injunction prevents only operation whereby a nuisance is created, and the plant is still operated, and from all intents will continue to be operated until it is proven a nuisance. Since this injunction was granted, the City of New York has awarded a new contract to the same company for the disposal of animals.

It would seem that greater results could be obtained if the public would look to the city for improvements and relief rather than to the Courts. "Control by anticipation" raises a difficult question and it is doubtful whether little would be gained, as compared to the results that could be obtained if the responsibility for securing the best method were placed on the municipality.

The developments to date in the elimination or non-production of odors have not resulted from any powers that could be granted pertaining to control by anticipation, but from adverse rulings and Court decisions, making it necessary to improve conditions in order to safeguard investments. There is a possibility that control by anticipation would have a tendency to retard development.

Little was known about conducting garbage reduction plants until the municipalities entered the field. Some advance has been made, from necessity

more than from actual study and attention to the development of the systems. When the same attention is given to this problem, as is given to many other municipal problems, and plants are constructed as permanent investments, advancement may be expected.

Rudolph Hering,* M. Am. Soc. C. E. (by letter).—The writer is pleased that the author has presented the results of his experience in regard to a subject which has been surrounded by much indefiniteness, ignorance, and prejudice. More facts based on reliable data were needed to form a fixed foundation for judgment, some of which the author has supplied.

Sentimental and personal perception play a large rôle in this subject; but thorough analyses and more impersonal data will help to reach useful conclusions.

Seven years ago, the writer presented a paper before the American Public Health Association on "The Prevention of Odors at City Refuse Disposal Works". As some of the data contained therein may contribute toward an explanation of this subject, a few of them will be repeated.

An odor is perceived by the stimulation of a small area, less than that of a one cent piece, near the upper end of the nasal duct. The normal excitation is caused by the impingement thereon of molecules of matter, either in a solid, colloidal, liquid, or gaseous form. That odorous air absorbs more heat by the addition of these molecules than pure air, was proved by Tyndall. That its specific gravity was increased sufficiently to retard the diffusion was proved by Tigerstedt, who likewise proved that odorous vapors are condensed by absorption at the surface of glass, paper, water, skin, and clothing. Odor molecules detach themselves from the surfaces by evaporation, oxidation, or by hydrolytic decomposition, and may be carried some distance in currents of air.

Some scientists have believed that only gases stimulate the olfactories. Others have demonstrated that liquids, such as sulphate of magnesium, also convey odors. Sharks have a well developed sense of smell.

Odors can be perceived more readily when the air is moist than when it is dry. A dry membrane in the nostrils cannot detect odors, nor does it always respond when the nostrils are completely filled with a liquid, even with eau de cologne. Some persons are incapable of detecting certain odors, such as vanilla, violets, and some faintly burning vegetable matters. The olfactories can tire of some odors and fail to respond; and yet at the same time they can perceive a sudden appearance of other odors. Nagel found that two or more entirely different odors may cause a composite odor, entirely different from either of the originals. Four grammes of iodoform can be made almost inodorous by 200 mg. of Peru balsam, as proved by introducing the two substances separately into the two nostrils. It was also found that the odors of paraffin introduced into one nostril will completely neutralize the odor of rubber, when a certain portion is introduced into the other nostril.

Prejudice has terminated in lawsuits, at times, as evidenced by complaints made of odors of burning garbage, when the actual cause was neighboring

* Cons. Engr., Montclair, N. J.

plants producing other offensive odors and when, in a new plant, smoke was seen to rise from the incinerator chimney and the plant was only being tested with preliminary coal fires.

Further, it must be realized that the odor particles are of different sizes. They are largest when in solid and smallest when in gaseous form. The odors escaping from incinerators are due usually to unburnt solid particles. As long as they drift in the air current from the chimney they will be noticeable, even for miles, in the form of, as it were, invisible clouds. The writer remembers such a case in Staten Island when suddenly in a clear sky and on an elevated bluff a strong burnt-garbage odor was apparent, brought by a gentle wind current straight from a refuse incinerator about two miles distant. It is evident that in discussing the question of odors many fundamental facts must be considered.

The intensity of odors is affected by temperature. If organic matter is burned at about 1 800° Fahr., no odor will result, nor is such matter offensive when it freezes. The most intense odor generally results between temperatures of 90 to 150° Fahr. Dryness likewise controls odor intensity, as has already been stated. In a dry climate, odors disappear rapidly.

Odors arising from gas particles which can easily decompose in the air, such as sulphureted hydrogen, disappear more quickly than when they arise from solid or colloidal particles.

From splashes or sprays the odors from sewage are increased when not only gaseous, but also colloidal, particles are thrown in the air, and drift quite a distance. Air from filters, sprinkled with putrefying sewage is often foul-smelling and is noticeable much farther than the air drifting over a quiescent sewage bed. Dilution of the foul air from the dryers at garbage reduction works, in the proportion of 1 to 500, has destroyed its odor.

When sewage containing odorous gases in solution or suspension, is discharged in streams, an odor has been perceptible near the water surface, even several miles below the plant. The odors after their escape, however, disappear very quickly, as they consist chiefly of gases, and not of colloids or solids.

As already stated, odorous particles suspended in the air, when adhering by absorption to surfaces of walls, clothing, etc., may persist for a long time. They can be removed by brushing and by washing down with blasts of compressed air directed against the surfaces. They can also be removed from clothing by exposure to sunlight.

From the facts known at present it is possible to reduce materially, and even prevent entirely, the odors arising from refuse disposal and sewage works, by a careful study of the specific conditions.

Olin H. Landreth,* M. Am. Soc. C. E.—A consideration of the travel habits of odors will readily indicate that they are lawless creatures, going with the wind which "bloweth where it listeth"; still, in their travels, they must be placed in the class of "short haul" rather than "long haul" traffic.

* New York City.

However, in this "short haul" travel, odors are no respecters of persons, nor of boundary lines, either private, municipal, or State, and, as a consequence, are occasionally found in jurisdictions to which they are not native, and even sometimes in adjoining States.

Since odors may be disagreeable, or offensive, or sometimes even seriously injurious to persons or property interests, they are frequently proper subjects for repression and abatement. When the occurrence of odors becomes serious enough to be a violation of law, whether common or statutory, that law may be invoked to repress those odors. Such legal repression or abatement is not always a simple or easy matter, even when the injury is suffered in the same legal jurisdiction as that in which the odors originate, but when the cause and the effect lie in different legal jurisdictions, and especially in different States, the problem of abatement is a still more serious one. It is not easy to reach across a State line and repress an odor.

Two instances will be cited where odors in their travels have ventured across State lines and have been abated by legal measures; in the first, by the legal remedies constitutionally provided and usually followed in such cases; in the second by creating, under what were rather unusual conditions, a novel form of legal relief, more expeditious and apparently as satisfactory in results as the usual form.

The first case is that of the fumes from the plants of two copper smelting companies at Ducktown, Polk County, Tenn., near the southern border of the State, which fumes were the cause of serious injury to property interests across the line in the State of Georgia.

The efforts of the injured parties to obtain relief by private litigation proved unsuccessful, and the State of Georgia finally sponsored the cause of its citizens who had suffered injury and, in October, 1905, brought an action for injunction against the two corporations in Tennessee responsible for the production of the injurious fumes, which, having destroyed all vegetation in sight in Tennessee, crossed over into Georgia and caused the injury to vegetation and animal life there complained of.

The defendant corporations were the Tennessee Copper Company and the Ducktown Sulphur, Copper, and Iron Company. As one of the parties to the litigation was a sovereign State, the case was necessarily brought as an original action in equity before the United States Supreme Court.

The Tennessee Copper Company's plant is about ½ mile from the Tennessee-Georgia line, and the Ducktown Sulphur, Copper, and Iron Company's plant is about 2½ miles from that line. The injuries suffered were from large volumes of sulphur dioxide discharged by both plants, originally from the ground-roasting of the ore, and, later, from the smelting furnace chimneys.

The copper ore used by both companies was a mixture of pyrrhotite and chalcopyrite carrying slightly less than 2% of copper and about 20% of sulphur. The Tennessee Company used about 450 000 tons of this ore per annum, and the Ducktown Company about 200 000 tons, and prior to the filing of the action, the two defendants probably discharged into the atmosphere about 240 tons of sulphur per day.

Like all original actions before the U. S. Supreme Court, testimony was taken before a commissioner, the hearings were widely separated and long drawn out, and at irregular intervals the Court itself heard motions, made rulings, delivered opinions, and, finally, in the case of the Ducktown Company, issued a decree granting a partial injunction which required the company to restrict the discharge of gases to the equivalent of 20 tons of sulphur per day during the growing season from April 10th to October 1st of each year, and to 40 tons per day during the remainder of the year.

In the case of the Tennessee Copper Company, the Court recognized a stipulation entered into with the State of Georgia, by which the State of Georgia agreed to refrain from asking a decree of injunction prior to October, 1916, on condition that the Copper Company would: (*a*) provide a fund annually to compensate injured parties in Georgia; (*b*) conduct its plant subject to inspection in specified ways; and (*c*) between April 10th and October 1st of each year limit the smelting of green ore to such quantities that the sulphur dioxide therefrom could be taken care of by its sulphuric acid plant when working at its normal full capacity. This arrangement between the State of Georgia and the Tennessee Copper Company still continues, and the case before the Court is continued indefinitely, no decree having ever been issued against this defendant. The awards of damages to Georgia claimants, the administration of the indemnity fund, and the compliance generally with the terms of the stipulation, are determined by a permanent commission of three members, of which one is chosen by the Tennessee Copper Company, and one member and an arbitrator or umpire by the State of Georgia.

During the past three years the defendant Ducktown Company has also been permitted by the Court to operate under this same stipulation conjointly with the Tennessee Copper Company. Each of the two defendants contributes annually to the indemnity fund a fixed proportion of the total awards made.

Thus, after many years spent in litigation before the U. S. Supreme Court, without including the previous period of litigation as private individuals, a reasonably satisfactory adjustment was finally reached.

The second instance cited of inter-state odors is that which Mr. Tribus simply refers to, namely, the case of the West End Association and the City of New York, Plaintiffs, *versus* The Barrett Company of West Virginia, The Corn Products Company, The General Chemical Company, The Midland Linseed Products Company, The Valvoline Oil Company, The Bulls Ferry Chemical Company, and Spencer Kellogg and Sons, Defendants.

This case is cited solely because of the very novel and original form of legal remedy invoked. During and before 1916, the defendant corporations which had manufacturing plants at or near Edgewater, N. J., on the west bank of the Hudson River opposite West 80th Street to West 110th Street, New York City, were, or at least some of them were, producing at their New Jersey plants offensive odors. These odors were blown across the Hudson River and greatly annoyed the citizens of New York City living on or near the east bank of the river opposite the plants in question. An action had previously been brought in 1915 by the Attorney General, representing the

State of New York, before the U. S. Supreme Court for an injunction to restrain the offending New Jersey corporations from discharging the objectionable odors. Beyond having some investigations made by the State of New York, the case was never prosecuted and still lies dormant. A strong local civic organization called the West End Association being already active in the locality affected by the fumes, took up the matter for the citizens. Instead of resorting to the usual litigation, either as private plaintiffs before the Federal District Court, or through the case already brought by the State of New York as plaintiff before the U. S. Supreme Court, as was done in the Georgia-Tennessee copper case cited, and in the New York *versus* New Jersey Bayonne fumes case, also referred to by Mr. Tribus, the West End Association developed the following form of relief.

An Act of the New York Legislature was procured, being Chapter 292 of the Laws of 1917, amending the general corporation law which constitutes Chapter 23 of the New York Consolidated Laws. This amendment, which became Article 9-A of the General Corporation Law, provides in Section 200:

"That any domestic or foreign corporation which shall so conduct its business without the State by the emission or discharge of dust, smoke, gas, steam, or offensive, noisome or noxious odors or fumes, so as to unreasonably injure or endanger the health or safety in this State [New York] of any considerable number of the people of this State, shall be deemed guilty of a nuisance, and the charter of such corporation if incorporated, or formed by or under any law of this State shall be deemed forfeited in the manner prescribed in this Section, or its certificate of authority to do business in this State [New York] if incorporated or formed under the laws of any other State shall be declared revoked and annulled in the manner prescribed in this Section and in either case shall not be revived except as prescribed in the next Section."

The law also provides the following procedure: Complaints shall be made to the State Commissioner of Health. He shall cause a copy thereof to be served on the corporation complained of, requiring that the matters complained of be abated, or that the charges be answered in writing within a time specified. If the charges are not thus satisfied, and if there appear to be reasonable grounds therefor by the State Commissioner of Health, he shall cause such charges to be investigated and shall fix a time for a hearing on such complaints. If the State Commissioner of Health after hearing and investigation shall find that such corporation is conducting its business without the State so as to injure or endanger unreasonably the health or safety in this State of any considerable number of people of this State, he shall file his findings in duplicate with the Secretary of State and with the Attorney General. A certificate of the Secretary of State giving notice of the filing of such findings shall be served on the corporation if domestic, or on the designated agent of a foreign corporation authorized to do business in the State, and thereupon the charter of such domestic corporation, or the right of a foreign corporation to do business in the State shall be suspended for thirty days. If at the expiration of that period, the State Commissioner of Health shall on further investigation and hearing render a finding that the nuisance is continued, he shall cause a notice of such determination to be served on the corporation or the designated agent of a foreign corporation,

and published once a week for two successive weeks in the official State paper. On the tenth day of such service and publication, the charter of the said corporation if domestic, or the certificate of authority to do business in the State if a foreign corporation, shall be deemed to be forfeited.

Any person who shall attempt to exercise any powers under the charter or the certificate to do business, which has been so revoked or forfeited, shall be guilty of a misdemeanor. If the charter of a domestic corporation has been forfeited, the Attorney General shall apply to the Supreme Court for the appointment of a receiver of the property, who shall have all the powers and duties, as far as is practicable, which are prescribed by Articles 10-A and 11 of the General Corporation Law. The Act also provides for the reinstatement of a charter or of the authority to do business, after the nuisance has been discontinued, and after a suitable guaranty has been furnished that the corporation will not longer maintain such nuisance.

Equipped with this new form of legal relief, the two plaintiffs in this action, representing and acting for a large number of private individuals and, perhaps, some firms and corporations, who were suffering from the odors, commenced proceedings in the summer of 1917 against the defendant corporations under the new law before the New York State Commissioner of Health. Two separate investigations were instituted: The first by the plaintiffs to establish in some detail the facts on which to elaborate the charges, to establish the general feasibility of abatement, and to base the examination and cross-examination of witnesses; the second, later, by the State Commissioner of Health to supplement the testimony developed at the hearings, on which to predicate the findings. Under the first investigation, the inspection of the plants of the defendant corporations clearly showed that the occurrences of odors causing offense in New York City could be grouped into two distinct classes: (1) those resulting from the usual, normal operation of the several plants, which, while fairly constant in character and volume at the plants themselves, nevertheless produced widely varying degrees of offensiveness in New York City, depending on the direction and force of the wind, and on the humidity and temperature of the air; and (2) those resulting from the occasional (originally frequent) lapses from normal operation of the plants, due to carelessness, interruptions, changes, etc.

The results of the litigation have been generally favorable. No charters or certificates of authority to do business in New York State have been forfeited, or even suspended. The immediate effect of the commencement of the proceedings was to diminish very considerably the frequency and severity of the spasmodic or occasional odors of the second class. In addition, the defendant corporations which were the worst offenders, have made extensive efforts toward abatement by experimentally introducing various means of preventing, absorbing, and neutralizing the fumes which cause offense in New York City. The results of these efforts, which are still being continued, have even thus far been fairly effective.

The hearings and investigations by the State Commissioner of Health are still continued at infrequent intervals, mainly for the purpose of determining the progress in abatement accomplished by the defendant corporations at the

plants, and of giving them further time in which to perfect their improvements. The West End Association, as well as the City of New York, maintains a vigilant and stimulating watchfulness of all proceedings and developments, and as plaintiffs are represented of course by counsel at all hearings and conferences. Thus, sanitary science has acquired another legal weapon with which to combat the nuisances arising from "some inter-state odors".

This form of legal remedy for injuries caused within a State by extra-State corporations is believed to be novel, and the present case of the West End Association and New York City against the New Jersey corporations is the first brought under the new law of 1917. It would seem that, with proper State legislation, this remedy should be applicable in other States and to other inter-state wrongs than odor nuisances, namely, to the cases of the diversion, to the interruption of flow, or to the pollution, of streams flowing through or past one State into or past another State, except in the case of streams which are debarred from State jurisdiction by reason of their being Federally navigable or otherwise subject to Federal jurisdiction.

It is hardly necessary to point out that the efficacy of this form of remedy, as far as it relates to foreign corporations, depends entirely on the degree of importance which the erring foreign corporations attach to the privilege of doing business in the State of which the complainants are citizens.

Andrew J. Provost,* Jr., Esq. (by letter).—The author has performed a real service in calling professional attention to the obstacles in the presentation before the Courts of specific, conclusive scientific evidence regarding odor nuisances.

In other nuisance evidence, such as relates to liquid wastes, the camera, volumetric measurement, laboratory analysis, etc., can be counted on to support and to lay the foundation for an expert opinion. In regard to the smoke nuisance, also, various supporting data are obtainable. On the questions of odors, however, the expert is unable to justify his opinion otherwise than by his sense of smell, which, to the Court, must appear little different from that of the lay witnesses.

If the expert could state that he was familiar with the process of the plant, the chemical combination of the gases discharged, and the constituents thereof which produce odors; that he had isolated from the atmosphere at certain distances from his plant odor-producing elements and had subjected these to analysis according to standard technique and had identified them as the same as those produced in the defendant's plant, and that he knew from observation and study that these elements are injurious or non-injurious, as the case may be, to the health and comfort of normal human beings, how different would be his position before the Courts. Possibly he will never be able to do this, but he can, perhaps, accomplish more for human comfort by working within the plant, by perfecting methods such as have been described by Professor Whipple, whereby the odors produced in the offensive trades will be absorbed, neutralized, or destroyed before reaching the atmosphere.

* New York City.

Some of the problems thus far studied would include: Garbage reduction works, slaughter-houses, piggeries, fish-rendering works, sewage treatment works, etc.

Garbage Reduction Works.—Many engineers began their experiences with the Barren Island plant, referred to by the author, and have appeared as friendly advisers or as hostile critics to its managers, at one time or another, during the past thirty years. It seems unlikely that another plant on such a scale will again be attempted and attentions and studies are directed to other types of plant, to better methods of operation, and to the evolution of more efficient operatives.

Slaughter-Houses.—The establishment of large abattoirs in the thickly populated centers, has resulted in intensified effort on the part of many operators to avoid the production and escape of odors. In some instances, chemicals which are quite effective in neutralizing certain peculiarly offensive odors, have been tried experimentally by the writer for the neutralization of other types of odors without much success.

Piggeries.—Offensive piggeries still exist, although it has been proven that the fault lies with the operator and not with the hogs. During the World War, many of the worst types of such plants were built, which defied regulation. Injunction proceedings were resorted to in some cases, but the general need for pork and the fact that the hogs consumed military camp garbage, secured immunity from successful prosecution.

Fish-Rendering Works.—Numerous large establishments exist, usually in quite isolated locations, for the extraction of oil from fish. The waste products consist of a slimy, liquid gurry and a comparatively dry scrap. The scrap is used as a fertilizer base and, if immediately treated with sulphuric acid to fix the ammonias, it is not particularly offensive, even if stored for considerable periods in large quantities. The gurry which cannot be discharged into streams without visible nuisance, is usually treated in tanks to precipitate the grosser organic solids. Fermentation results in an odor indescribably offensive. As the result of complaint and State action in some cases, condensers have been erected, which are effective in disposing of gurry wastes without the escape of odors and, at the same time, have conserved an appreciable part of the available nitrogen. A plant of this kind, which was threatened with an order to suspend operations, was modified by the writer to such an extent that complaint was withdrawn.

Sewage Treatment Works.—The cause for odors in and about a sewage treatment plant usually arises from ignorant, incompetent, or careless operation. There are, however, certain odor-producing processes which can be largely controlled by the designer. It is known that the odors which travel farthest and which cause the most complaint are produced by spraying the effluents from sedimentation tanks in the operation of open sprinkling filters and by the open lagooning or drying of sewage sludge. In certain cases of this kind, nuisance odors persist to distances as great as ½ mile or more. The writer has been able to control these odors so as practically to avoid all complaint, even in plants located in the midst of settled communities, by

housing the sprinkling filters and the sludge bed areas. The gases are then either re-absorbed by the effluent liquid or are discharged in such uniform, small quantities as to be substantially unappreciable.

Engineers who design apparatus for handling and disposing of offensive products and wastes are justified in demanding the highest efficiency in operation and regular supervision of the plant.

Odors travel usually in the direction of the wind and are frequently unnoticeable in other directions. In the absence of wind and vertical atmospheric currents, with a low barometer and with fog or mist, there appears to be a tendency for the odor-carrying gases or vapors to combine with the atmospheric moisture and to travel with it, close to the ground, until gradually released.

The plea made by the author for regulation of offensive trades by imposing restrictions on the installation of equipment and proposed methods of operation, instead of waiting to abate a nuisance after it is created, appears to be sound and deserves careful consideration.

Research along many lines is required before sufficient knowledge of handling putrefactive and fume-producing products is acquired to permit experts to go before the Courts and satisfy them beyond a reasonable doubt that as planned a certain project will or will not create a public nuisance.

In cases where the nuisance has been created, the Courts have held, as cited by W. H. Dittoe, M. Am. Soc. C. E., in his monograph, "How to Control Nuisances from Offensive Trades", that:

"It is no defense to an indictment for maintaining such a nuisance, that the business, trade, or occupation which occasions it is a useful one, or that it is really a public benefit, contributing largely to the enhancement of the wealth, prosperity, or commercial importance of the community, or that it furnishes, on the whole, a convenience to the public which more than counterbalances the detriment it occasions. For if it is in reality a nuisance or operates as such on the public, no measure of necessity, usefulness, or public benefit will afford a justification for maintaining it. Nor is it any defense to show that the business is carried on in the most prudent and careful manner possible; that the most approved appliances known to science have been adopted to prevent injury. The question of care is not an element in this class of wrongs; it is merely a question of results, and the fact that injurious results proceed from the business, under such circumstances, would have a tendency to show the business a nuisance *per se*, rather than to operate as an excuse or defense, and the Courts would feel compelled to say that, under such circumstances, the business is intolerable, except when so far removed from residences and places of business as to be beyond the power of visiting its ill results on individuals or the public."

No doubt there are special and unusual cases of odor nuisance which are entitled to a somewhat different viewpoint. The writer has in mind some experiences resulting from his service as sanitary expert during the construction of the Catskill Aqueduct. For a distance of about ten miles, this work ran through the Croton water-shed, which at that time was furnishing most of the water supply for the City of New York. Several thousand workmen were employed, most of whom were housed in regulated labor camps. When it was contemplated to commence this part of the work, the decision was reached that all human wastes must be kept from the streams and water-

courses and that the most effective means would be secured by incineration. Removable, water-tight receptacles were provided for the entire line of the work. These were collected daily and their contents burned in various types of furnaces. The strange, original character of odor produced was at the first experience quite startling. The gases, due to their high temperatures, were very volatile and were most noticeable at considerable distances and usually on higher ground. By carefully selecting the sites of the furnaces, it was generally possible to minimize the odor nuisance, and although the practice of night soil incineration was largely extended to the other sections of the work, throughout a length of about 100 miles, very little complaint was made. Possibly, this was because people felt the annoyance was only temporary and that its presence was justified largely by the necessity of safeguarding the sanitary condition of the streams.

Had the complaints been more numerous and severe, it is quite possible that means might have been provided for reducing the odors by smoke consumers of suitable type. During the five years' occupancy of the Croton watershed not a case of typhoid developed among the force employed, nor at any time was there any suggestion made that the sanitary quality of the water supply of the city had been impaired.

If injunction had been sought against the odors from these incinerators, it appears highly probable that neither the Courts nor any expert board would have felt that the alleged discomfort caused by their operation was paramount to the necessity of safe-guarding the water supply of a great city.

Garbage and other putrefactive organic wastes must of necessity be destroyed in the most rapid manner practicable, and this necessity imposes the obligation to devise apparatus and methods of operation which will avoid, as far as possible, cause of offense and discomfort to individuals and the public.

Robert Spurr Weston,* M. Am. Soc. C. E.—The speaker will confine himself to a discussion of three odors: First, those given off by sugar-house waste; second, those given off by wastes from the manufacture of lactic acid; and, third, those produced by oil refineries.

In the manufacture of sugar, cane juice is expressed; the water is evaporated from it in multiple effects and vacuum pans, and cane sugar is obtained from the concentrate by crystallization. The distillate which contains considerable sugar, is usually wasted, together with some organic matter and some press-cake water containing organic matter, including various gums and sugar. Cuban sugar operations are conducted in the dry season and the wastes are discharged into lagoons or dry arroyos. These wastes begin to ferment even before they leave the sugar-house, which results in a vile odor from the hydrogen sulphide mixed with acid and the organic compounds containing sulphur; there is no odor much more disagreeable. Starch factory waste is nearest in the odors given off; but most of the starch factories, are located in cooler climates, and the nuisance is not so great. Those who have been through Cuba have experienced this odor from sugar-house wastes,

* Cons. Engr. (Weston and Sampson), Boston, Mass.

and have noted the effect of the waste on the streams. The remedy for this odor lies in a better method of waste disposal, which has not yet been attained.

Lactic acid is made by hydrolyzing starch and fermenting the product. The principal source of starch is vegetable ivory, a dense tuber from Africa, which ferments, producing lactic acid. This acid soon accumulates, and inhibits the growth of the bacteria of fermentation. Fermentation starts again after lime is added. In a short time, the process reaches its limit, and the lime and organic matter are precipitated. The precipitate is removed by filtration, and the filtrate containing the lactic acid is concentrated in vacuum pans. If the various wastes from this process are discharged into a lagoon or a stream where dilution is insufficient, they ferment with the production of butyric and other organic acids, hydrogen sulphide, and also compounds of sulphur and organic matter. Between New York City and Boston, Mass., a plant was started during the World War, and continued until it was put out of operation in 1919 by the action of a town board of health. The odor from the untreated waste was observed at times 2½ miles away and white lead paint within a radius of ¼ mile was turned black by the sulphur gases given off.

During 1921, the speaker had to deal with the odors produced by oil refineries, which odors Mr. Gage has also described. The source of fuel oil, is crude petroleum, and refineries are using more and more Mexican oil which is rich in sulphur. This oil is split by distillation into fractions, the lighter fractions being kerosene and gasolene. A refinery sometimes produces lubricating oils, gas oils, fuel oil, and, in addition, the heavier products, such as paraffin, asphalt, and coke. For many generations, the people of Massachusetts have been accustomed to the odors produced by the drying of the descendants of the "sacred codfish"; but oil odors are new and terrifying. Not only were objections made to the odors from the newly constructed refineries, but the smoke produced by the burning of oil was also a cause of complaint. Coal furnaces which had been hastily altered to burn oil, frequently produced a sulphurous smoke, particularly on Monday morning when late firemen tried to make up for lost time. Where the vapors and smoke were uncontrolled, conditions became so that people closed their windows at night to shut out the bad smells, and cases of nausea and vomiting were common.

Economic conditions demanded oil for fuel, and more refineries were completed. Locations were often unwisely chosen, and neighboring districts were gassed until the residents rebelled. This intolerable condition resulted in a demand for the cessation of either the odor or of oil refining. Certain refineries have stopped odor production; others have been forbidden to perform certain offensive operations, such as making coke, etc.

The means taken by certain refineries to control the production of odors have not been successful, their failures having been due to poor engineering, and especially to poor management, rather than to inherent difficulties. The most successful plants have either put all the refining apparatus, including tanks, under a vacuum system, and have passed all the gases through a fire, or subjected them to treatment by washing with some of the raw material going through the process; that is, the counter-current principle has been

applied by washing the distilled gases with the raw material about to be distilled.

The washing of gases in water simply transfers the odors to the water which has at least as much tendency to give off gases to the air as the oil itself, and very often the discharged wash water simply transfers the odor to another location.

In observing an odor-producing plant, the first effort should be to make the adjectives which have been used so generally to describe odors, to mean something. One way of doing this is to have the smelling systematic, that is, locate the odors from the leeward side of the plant, in order to determine the distance an odor can be detected under various air conditions. By plotting the observations day after day, one can obtain a graphical representation of the characteristics of any plant for odor production.

In order to make the record of one gas more complete, the speaker's firm, with the aid of the Wallace and Tiernan Company, Incorporated, adapted what is known as the Palmer dust apparatus for measuring the hydrogen sulphide in the air. The method was more delicate than the human nose, which, ordinarily, was able to detect about 1 part of hydrogen sulphide in 10 000 000 parts of air. This is a low concentration, but with the apparatus referred to, as little as 15 parts of hydrogen sulphide in 1 000 000 000 parts of air could be detected or about one-eight of the quantity which the observers could smell.

This Palmer spraying apparatus is really a nebulizer, which absorbs gases passed through a nebulized absorbent. It consists of a U-tube with a condenser, through which air is drawn by a fan and connected with the fan is a Venturi meter. It was customary to draw 100 liters of air through this device in 4 min.; 100 liters is about equal to ½ bbl., so that a considerable sample of air was tested at one time. By having three assistants cover the 24-hour period, an accurate record of the hydrogen sulphide content was made. In the tests, 35 cu. cm. of $\frac{\text{Normal}}{10}$ caustic soda was put in the U-tube. The air was then drawn through the liquid which nebulized and absorbed the gas. The caustic soda solution was then washed out of the U-tube and the hydrogen sulphide determined by titration with $\frac{\text{Normal}}{1\,000}$ iodine solution, using starch for an indicator.

This apparatus was tested thoroughly, and it was found that it would absorb all the hydrogen sulphide from the air passed through it and also any sulphur dioxide. Furthermore, it had an accuracy of ± 15 parts in 1 000 000 000.

It is well that engineers are attacking this odor problem. Sanitary engineering was a combination of bacteriology, chemistry, climatology, hygiene, and civil engineering. This new work which the speaker hopes will not be given a new engineering name, includes also meteorology, physics, and chemical engineering. The concentration of population and the rising esthetic standards will make work for engineers in these new fields.

ALEXANDER POTTER,* ASSOC. M. AM. SOC. C. E.—A few years ago, the speaker was asked to investigate the existence of odors in certain sewers. In the course of the investigation, the question of the travel of odors through sewers was studied and some interesting results were obtained both as to their direction and rate of travel. It was found that offensive odors were given off thousands of feet down stream from the point of emanation.

About twenty years ago, the speaker constructed a system of trunk sewers for a number of municipalities in New Jersey, including portions of the Cities of Elizabeth, Newark, Summit, Irvington, South Orange, West Orange, Millburn, and other places. During the construction of this system of trunk sewers, he also had supervision of the construction of most of the lateral systems within the confines of the various municipalities. Flush tanks were recommended and constructed throughout most of the lateral systems, but the main house-trap was omitted through most of the district, ventilation taking place through the house-risers.

In only one municipality did the local health authorities insist on the use of the house-trap, but, in order to economize on water, flush tanks were not put in service, dependence for flushing being placed on the use of a hose stream in the upper end of the sewers. As often happens under such conditions, the sewers were not flushed for weeks and months at a time.

Although the conditions as to capacity, flows, grades, and velocities were similar throughout the entire district, complaints were made from time to time about odors along the Joint Trunk Sewer through this municipality, which was built at joint expense to provide sewerage facilities for certain outlying districts. Repeated examinations of the Joint Trunk Sewer disclosed nothing that would justify the conclusion that the odors in this section of the trunk sewer had their origin therein.

The speaker's contention was that the odors were caused by unsanitary conditions of the lateral sewers, from which odors found their way down grade into the Joint Trunk Sewer. The air space above the flowing sewage in the Joint Trunk Sewer was proportionately very much less than the combined air space in the many contributing laterals; therefore, as the foul air found its way down stream to the Joint Trunk Sewer, it was forced out through the manholes, the quantity fluctuating with the volume of sewage during the different hours of the day.

It is a popular belief that the direction of the flow of sewer air is up hill. Experiments were made to substantiate the theory that the flow was down hill. At certain points along the Joint Trunk Sewer and the lateral sewers, smoke balls were introduced and the direction of the smoke was noted. The velocity of the smoke and its volume passing down the sewer, together with the velocity of the flow of the sewage, are indicated in Table 3.

The speaker believes that the controlling factor in directing the flow of the air down stream is the motion imparted by friction with the flowing sewage. Down-stream components are thus produced sufficient to carry the air in the direction opposite to that which might reasonably be expected.

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* Cons. Engr., New York City.

Table 3 records velocities as high as 2.38 ft. per sec., which were sustained for distances of ½ mile in certain sections of the sewer.

TABLE 3.—SUMMARY OF EXPERIMENTS TO DETERMINE MOVEMENT OF AIR IN SEWERS, OCTOBER, 1916.

No. of experiments.	Sewer:		Average Velocity:		Ratio, in percentage.	Movement of air, down stream, in cubic feet per day.
	Size, in inches.	Grade, per hundred.	Sewage, in feet per second.	Air movement, in feet per second.		
1	8	1.9	1.6	0.32	20	29 300
2	12	0.25	3.2	0.24	8	12 100
3	10	2.3	4.0	1.50	35	60 100
4	10	0.84	3.4	1.20	35	125 000
5	10	0.84	3.4	1.12	33	121 000
6	10	6.5	2.7	2.38	88	89 700
7	15*	0.56*	3.1 main line 3.0 relief	1.94 0.29	63 10	175 000 41 600
8	18-20	1.56-0.28	4.3	1.06	25	112 000
9	12	0.75	3.1	1.30	42	58 300
10	15	0.40	1.82	0.29	16	26 800

*Main line only.

As a result of these experiments, the speaker not only succeeded in having the sewers cleaned more frequently, but also had the traps eliminated so that gases forming in the sewers were freely vented to the atmosphere at each house connection, and thus the nuisance was abated.

CALEB MILLS SAVILLE,* M. AM. SOC. C. E. (by letter).—As a boy, the writer well remembers the smell and the choking sensation, experienced at night, when a strong east wind blew up the river valley in which his home was located. This wind was followed in the morning by a yellow haze, which left a dark discoloration on the houses in the lower part of the town.

The conditions producing the odor mentioned were due to emanations from a chemical manufacturing plant about 4 miles to the east, of which sulphur products were the principal industry.

Subsequently, in another city, a large gas plant belched out its fumes at night to such an extent that it was impossible to sleep, at a distance of 4 or 5 miles away, on certain nights when wind and other atmospheric conditions were in conjunction. Happily for the inhabitants who are obliged to live in these places, both these nuisances no longer exist. Their disappearance, however, was not due so much to the regard of the plant directors for the comfort of the neighboring communities, as it was to the results of experimentation and advance in chemical science, which results indicated that valuable products were being wasted. Perhaps, a solution of the problem of community preservation from nauseating odors, would be to put industrial chemists at work to find a process which will dangle the lure of the dollar before the eyes of the plant manager. The possibility of such attainment might cause him to seek eagerly to conserve his filthy smells in order to exchange their cause for money. Possibly, we have not yet advanced far enough along scientific lines to cope successfully with many stages of this subject, and it may be that later developments will disclose the fact that all these odors are as yet

* Mgr. and Chf. Engr., Board of Water Commrs., Hartford, Conn.

unrecognized indications of a present waste which, when properly treated and conserved, will yield products too valuable to be overlooked. For example, it was only a comparatively short time ago that the gases of combustion were wasted, yet, although people were susceptible to the coal-gas smell which was more or less noisome to some, it was not recognized until recently what waste was going on in unconsumed fuel.

The author remarks on the hopelessness of attempts at the mathematical measurement of odors and rightly mentions the disagreement as to the character and intensity of the same odor. Although this is true, as applied to those who are untrained, it is equally true that the organs of taste and smell are susceptible of marvellous development. The practical agreement of several trained observers as to the intensity of odor of a water and the almost uncanny unanimity of opinion of several tasters of coffee, tea, and spice is sufficient indication of what it is possible to attain.

In this connection, also, the writer would like to call attention to the unique and valuable work of George C. Whipple, M. Am. Soc. C. E., in classifying the comparatively faint odors of potable water. On page 18 of his book on the "Value of Pure Water", Professor Whipple gives a diagram from which he says "one may calculate what may be called the esthetic deficiency of water". The curves of this diagram are built up "by adding together the percentages of objecting consumers", 100% being an odor to which everybody would object. In his discussion, Professor Whipple recognizes the difficulties of classifying odors and gives three separate equations for satisfying as many conditions.

Although these curves relate, as stated, to the comparatively faint odors of a potable water supply, it may be that a similar classification can be worked out for the odors of various industries. After some study, a scale may be arrived at, whereby it will be practicable to make more definite statements as to the probable number of people in the area exposed, who will be affected seriously by a certain odor, and thus decide whether or not the odor is a nuisance. It is well known that some persons are especially sensitive even to slight odors; however, if only a small percentage of the whole number are affected, the condition could hardly be called a public nuisance and a subject for drastic action.

Whatever conclusions are arrived at, in any individual case, it must be recognized that the solution of the problem lies in a proper balance of many conflicting elements, and that, very likely, the matter may resolve itself into a decision as to the degree of odor permissible rather than total elimination. The standard probably would be based on the reasonable use of the air, the possibilities of practical remedial action, and the economic value of odor elimination, as opposed to air pollution by the discharge of noxious or ill-smelling vapors.

Proceeding along this, or a similar, line, it would seem that after a time a sufficient number of precedents would be established, so that it would be possible with some degree of certainty to bring forward such evidence of polluted air and its results, as affecting persons and property, that the Courts would be willing to apply measures for relief. For this purpose, however,

trained observers are required, for the ordinary individual is incapable of differentiating as to intensity of odor and, indeed, it is probable that a large proportion of the people of a community are unable, quickly, to determine whether it is taste or smell which offends them.

Proceeding along the lines which have been suggested, it is probable that a valuation curve, or rather one of depreciation could be worked out similar to Professor Whipple's valuation of attractive water, from which the financial loss to a community, as well as the personal annoyance of the inhabitants, could be determined. Such evidence, if it were available, would be much more effective in abating a nuisance than many individual complaints of a more or less indefinite nature, which when investigated often become of little value, because of differences in the personal equation of the observer.

In general, the matter of elimination of disagreeable odors seems to be strictly analogous to the pollution of natural watercourses into which sewage and waste are discharged by the method of disposal by dilution. Some of the methods found to be helpful in correcting unbearable conditions in regard to water pollution may, perhaps, be useful in dealing with the pollution of the air.

George A. Soper,* M. Am. Soc. C. E. (by letter).—The author has presented another question, which affects life and comfort intimately and continually and on which there is a deplorable lack of information. It is surprising to note the number of these questions, and it is well for engineers to discuss them. Such discussions may not always advance the subject very far, but they, at least, bring together many of the more firmly established facts concerning it, and are likely to indicate the directions in which more practical information is needed.

The basis on which the paper is presented is interesting. A great deal of scientific discussion could have been abstracted from encyclopædias and other sources, but the author seems purposely to have avoided this form of exposition and to have directed his inquiries to the observations of engineers, apparently in the hope of obtaining first-hand information on a number of specific subjects which he has carefully stated.

The paper was written in a more or less humorous vein, for in no other way can the title and some of the theories which have been put forward be so well explained. It is, of course, as well known to the author as to any one that odors do not have "habits", that they do not in strictness "travel", that there are no such things as "smell atoms" and that to seek to kill one odor with another is almost always bad sanitary practice. This procedure has been extensively practiced, especially with toilets, with the result of tainting the air of hotels, theaters, and clubs and rendering the presence of ill-kept water-closets unduly noticeable in subways, railroad stations, and boats. It is hoped that this practice will not be attempted, on a large scale, out of doors. The general atmosphere is a large space, as the writer once found when he was called on to get rid of the odor produced by the fermentation of more than 100 000 bushels of wheat, which had been water-soaked in a flood and, later, exposed to a hot sun.

* Cons. Engr., New York City.

The subject which the author states he particularly wants discussed is the distance from sewage and garbage plants to which an offensive odor may travel and cause serious annoyance. A brief and direct answer to this question cannot be made. The subject is too indefinite; nothing is fixed about the problem. Every element in it is variable. In a general way, it may be said that odors will not travel far unless under exceptional circumstances as to the quantity of the odorous substance and favorable states of the atmosphere; but this statement lacks the very precision which is desirable.

Before much progress can be made toward definiteness, it will be necessary to define the terms used. It must be agreed as to what shall be meant by the word "offensive", and the word "annoyance" must be defined. These words are difficult to define, as they represent mental states which, in turn, are produced by sensations. They are reactions which result from reactions. They are subtle and are likely to be affected by conditions which cannot be discovered to have any necessary connection with the problem. The final effect which may result from the operation of a set of known conditions at one time, may be different from the effect which the same conditions may appear to produce at another time.

For these reasons, that which is disagreeable to one person may not be so to another; that which is unpleasant on one occasion may not be so on another. It often happens that a person at first objects to an odor to which he later becomes indifferent; and he may become actually fond of it in time. The odor of cheese affords an example; tobacco smoke is probably another. People are not fond of these odors at first, however much they may like them later. It is not the odor, but what it suggests, which is the important thing.

If the odor suggests the existence of something which is dangerous, much more objection will be made to it than if no alarming association is aroused. The element of danger, however, need not be greatly in evidence. Odors may suggest conditions that are disturbing to one's sense of comfort, of well-being, and when this sensation results, the objection raised may be emphatic. As regards emphasis, it is well to remember that some persons by temperament, speak loudly, although only slightly annoyed, whereas others are truly long-suffering. In other words, the annoyance produced is not to be measured by the noise which is made about it.

There is little doubt that physiological actions are stimulated by odors, and it often happens that these are not only not within one's control, but they may be beyond one's knowledge. It seems reasonable to suppose that a hospital placed in an agreeable pine forest may have a different effect on its patients than one located in a malodorous part of a city; and what is true of people out of health in this case may apply in lesser degree to those whose health is normal. It is fair to suppose that the manifold unfavorable conditions of city life, among which odors occupy no inconspicuous place, have a decided influence on most of the people who encounter them. Cities are by no means the healthy places which some have been led to suppose.

The late Professor William T. Sedgwick used to tell a story to illustrate the fact that even strongly disagreeable odors may seem pleasant and beneficial according to one's idea of their composition, origin, and effects. The school

teachers of Massachusetts were accustomed to meet in annual convention during the summers in the City of Boston. A feature of these annual gatherings was a trip on the harbor, with a stop for dinner at one of the shore resorts and an hour or so of leisure on the rocks. On these excursions, Dr. Sedgwick would say, the little schoolmistresses would be fairly exalted by the odors greeting them from the sea. They would climb down to water's edge, fill their lungs with long draughts of air and delightedly pronounce the magic word "ozone". "But", Professor Sedgwick would conclude, "You and I know that what they smelled was rotting sea weed. If they had known what caused the smell, they would have fled the place, and lost the benefit which the air and their ideas of the fresh sea breeze were giving them."

It is hoped that, for the benefit of sanitary science, further study will make the detection, measurement, and recording of odors more definite than it is to-day, but it seems too much to expect that the time will ever come when this subject will be put on the more satisfactory basis which exists with respect to the evidence of some of the other senses.

The odors from certain gases, dusts of various composition, and vapors of known chemical composition can be investigated with an approach to definiteness, but even when the amount of the irritating substance present in a given volume of air is known, no great progress will be made until the reaction of the average person toward the odor can be ascertained. To procure this information may not be possible, but it is a line of effort which certainly is worth further study.

This method of research was not considered practicable by the writer in his studies of subway odors.* The odors were mixed, as is often the case in practical problems. They were of various kinds, and for a long time their sources were in doubt. It was not until experiments were made to duplicate them, that the origins of the odors could be found. Briefly, these sources included the trap-rock of which the roadbed was composed, the hot brake-shoes, the lubricating oil, which was lavishly used when the road was first opened, the cement, which was used to fasten the tiles in place at the stations, and the great quantity of iron dust, which was produced in the operation of the cars.

It may be worth while to call attention to the fact that no relation exists between the offensiveness, or suggestiveness, of an odor and the distance to which it may be carried. For example, the River Thames at times is odorous from sewage throughout a distance of 25 miles below London Bridge, yet the smell does not travel far beyond the river banks. The East River, Harlem River, and Hudson River, at places, are very offensive, but it is only on particular occasions, fortunately not frequent as yet, that smells are carried more than a few hundred feet inland. It is reasonable to suppose that the malodorous zone will increase in time, if the harbor continues to be neglected, until the region about the shores will be continuously affected; but the waterways themselves will be highly offensive before a great deal of the smell is carried far toward the center of the city.

* "The Air and Ventilation of Subways", Wiley & Sons, N. Y., 1908.

There are frequent times, when the greasy smell of sewage is noticeable along miles of the Manhattan water-front. It is frequently detectable over a large part of Riverside Park and on the drive which borders the eastern limit of the Park; and it is unpleasant in some of the other parks which border on the harbor. Some of the recreation piers have sewers discharging directly under them, and the odors which assail the thousands of visitors at the times when these breathing places are most needed, are nauseating to those who have not become accustomed to them. In all these cases, however, the odors do not travel far. They annoy a great many people, not because they are carried a long way to the places where they are noticed, but because the people go where the smells originate. They go there to get fresh air and to enjoy the provisions the city has made for their health and enjoyment. By thus discharging the sewage, the city does much to destroy the attractiveness of these places which it has provided for the public welfare.

An incident has recently come to the writer's attention which shows that the odors of sewage are sometimes carried more than 1 000 ft. inland from the polluted waters which surround Manhattan Island. One night in the summer of 1921 a resident of East 86th Street was awakened by an odor which at first seemed to be due to an escaping gas jet. On opening the window, the smell became stronger, and it was observed that windows on the opposite side of the street were being raised by persons whose sleep apparently had also been disturbed, and people collected on the sidewalk to discuss the situation and decide what, if anything, was to be done. Investigation showed that the smell came from the East River, being carried by a gentle breeze. The night was warm and damp; the tide was low. The odor was uncommonly strong, which may account for the fact that its cause was not at first generally recognized. In some neighborhoods, the smell of sewage from the East River is of considerable interest, for it is said to furnish the citizens with a means of foretelling the approach of a storm.

Surgeon General Sternberg used to say that disagreeable odors should be interpreted as danger signals, for although they are unreliable guides if followed blindly, they are capable of doing great service in suggesting conditions which ought to be investigated.

Louis L. Tribus,* M. Am. Soc. C. E. (by letter).—The discussions confirm the writer's opinion, that only through the development of trained observers working along the same lines can intelligent classification be reached and anything approaching mathematical precision be attained.

The writer did not attempt any real definition of what constitutes differences between offensive and agreeable odors, or between those that may be harmful and those that are benign. He merely sought to outline the subject to which others might add data and from which, with later additions, helpful facts and conclusions might be taken. The results have warranted the hope.

Mr. Hansen enlarges on the point that certain industries must be considered essential, and thus be tolerated, while all effort should be made to cure the attending evils; also, that odors are most apt to be the barometer by

* Cons. Engr. (Tribus and Massa), New York City.

which the public rates them. He offers zoning as one of the means of relief, but not as a cure.

Professor Whipple discusses interestingly, the personnel of odors themselves and treats the subject along the following logical lines: (1) Physiology; (2) relation to health; (3) meteorological factors of travel; (4) governmental control; and (5) prevention.

Temperature has quite an influence, due to volatilization of the odor elements, and humidity plays an important part; and those discussing the paper naturally mention wind movement as the important element in dissemination.

Professor Whipple calls attention to the fact that frequently odors are an indication of unwholesome conditions, but that unwholesome conditions must not always be thought to be absent when definite odors are not present—carbon-monoxide is virtually odorless, yet very deadly, and is given off in large quantities during the combustion of most fuels. Governmental control cannot well step in until nuisance exists or can be clearly anticipated; and such control then should be handled with great circumspection, yet as the writer suggested, and to which others have agreed, it might be secured in part by intelligent official advice, prior to the act.

Prevention is a field for intensive study, and is one that has occupied the attention of many a factory chemist, spurred in some instances by public sentiment and wrath, but often by the earnest desire of the plant management to do better things.

Mr. Gage tells of some interesting observations in attempts to control the offensive odors from certain oil refineries in Rhode Island, the presence of which was greatly desired as helpful industries, yet the operations of which were not the most agreeable. The fact was brought out that, in these cases, hydrogen sulphide and other sulphide gases were the chief offenders, the Mexican oil that was used containing more than 4% of sulphur.

It is difficult to realize the enormous quantities of these gases, certainly not helpful and highly odoriferous, that are given off daily in refining the much needed oil.

Mr. Osborn, out of his long experience in the study of the design, construction, and operation of garbage works, corrects the writer in certain references to garbage plants, and is probably justified in so doing. He devotes most of his discussion to the action of garbage reduction works and the causes of the dissemination of their odors, which he admits are offensive when not controlled.

The point which he emphasizes as of greatest importance, is not so much how far odors do or can travel, but how can they be prevented from traveling, a most pertinent conclusion.

Mr. Hering gives some physiological facts as to the perception of odors and their neutralization; that liquids make good carriers and that careful study makes possible almost complete elimination in cases of refuse disposal and sewage works.

Professor Landreth has given quite complete data on certain interstate litigation of prime importance, and New York legislation that has recently

been helpful in abating certain undoubted nuisance cases, where odors of offensive character affected humanity and property at great distances.

Mr. Provost emphasizes some of the difficulties in the way of expert testimony and discusses certain specific types of offensive works, where publicity and, in some instances, drastic action, has secured the abatement of the nuisance, and also where a temporary nature has permitted operations, when permanence would have demanded cessation or change.

Mr. Weston discusses particularly the odors from sugar-house wastes, manufacture of lactic acid, and oil refining, and gives enlightening description of the processes and of testing apparatus designed especially to detect and standardize the odors given off, so as to make records that should be comparable and intelligible.

Mr. Potter contributes certain air-movement tests in sewers, looking to acquisition of knowledge as to the occurrence and behavior of sewer gases.

Mr. Saville rather looks forward to such scientific study, that curves and scales may yet be developed, that will enable far greater approximation of effects under standardized conditions, so that trained observers may even present data, that will be reasonably conclusive to the Court.

Dr. Soper emphasizes, as did the writer, the intangibility of odors; that some which at first might be offensive, later become tolerated; that offensiveness may not prove real danger, but should be a notice to take warning; that they have no fixed "habits of travel"; and gives from his experience in research, particularly in New York's subways and in its harbor, some interesting items.

In general, it is agreed that a study of odors and their habits of travel would be of great economic service, and that on the engineer, devolves that duty.

The writer feels justified in having brought forward the topic, believing that, from it, more of helpful facts will be gathered and promulgated for the betterment of general living and, perhaps, the pocket-books of engineers.

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Paper No. 1487

STREAM POLLUTION AND SEWAGE DISPOSAL*

A SYMPOSIUM

By Messrs. George T. Hammond, Kenneth Allen, John F. Skinner, W. L. Stevenson, Earle B. Phelps, T. Chalkley Hatton, Langdon Pearse, and W. H. Dittoe.

With Discussion by Messrs. Harrison P. Eddy, J. Frederick Jackson, F. A. Dallyn, W. F. Wells, Alexander Potter, Edward S. Rankin, S. John Scacciaferro, Glenn D. Holmes, Rudolph Hering, C. M. Baker, P. H. Norcross, Leonard S. Doten, John A. Griffin, and Clarence E. Keefer.

* Presented at the meeting of November 16th, 1921.

TANKS AND FINE SCREENS FOR TREATING SEWAGE

By George T. Hammond,* M. Am. Soc. C. E.

In considering the title of this Symposium, "Stream Pollution and Sewage Disposal", and what should be said about tanks and fine screens, the speaker's first thought was that perhaps the title should have been "Sewage Disposal and Stream Pollution", for observation of actual conditions rather tends to justify the statement often made, that sewage disposal is the main cause of stream pollution; this is sometimes the case even where treatment plants have been established. Not to mention instances known to most engineers, in which sewage is "disposed of" without any attempt at treatment, examples of American idealism and carelessness in sanitary works are sometimes found, in which elaborate and costly plants are constructed, and do not function because of ignorance or neglect on the part of those whose duty it is to care for their proper operation—sanitary engineering efforts conceived in faith, which, however, will not function on faith alone.

Engineers are familiar with the various forms of tanks of which quite a number are obsolete, and of fine screens that have been in use for many years in sewage treatment works, some of which are also obsolete. There has been an evolution from the ancient and odorous cesspool to the sedimentation and separate sludge digestion tanks of the present. Experience and experiments go far to prove that the latter afford the most satisfactory method of tank treatment, securing a high removal of suspended solids and complete digestion of the sludge, without much danger of a nuisance. Much experience with the design and operation, as well with overloading and abuse, of such tanks was gained during the World War, and the interested student can inform himself of many data from several important papers since published, of which the speaker will only mention the one presented by Leonard S. Doten,† M. Am. Soc. C. E., before the Society.

The speaker will not attempt any discussion of the various forms of tanks and fine screens so fully described in textbooks and so frequently seen in treatment plants abroad, but will confine himself to those which seem to have demonstrated their usefulness in ordinary American practice.

One of the latest extensive projects in sewage treatment is the proposed enlargement of the plant at Baltimore, Md., concerning which an interesting paper‡ has appeared recently. The following statement is quoted from that paper:

"Everything points to the adoption of sedimentation tanks in conjunction with sludge digestion tanks as the most logical solution of the problem for Baltimore conditions. None of the many troubles that have to be confronted in the operation of Imhoff tanks is found in treating sewage with sedimentation and sludge digestion tanks. No foaming is met with; no time is spent squeegeeing; no scum has to be removed; no skimming of tanks is required; and no uncertainties of operation have to be considered."

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* Cons. Engr., Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 337.

‡ *Engineering News-Record*, Vol. 87, p. 654.

The experimental work on sewage treatment in Brooklyn, N. Y., extending over more than five years, appears to justify the conclusion reached in Baltimore as to the success of sedimentation and separate digestion, except that much difficulty was not found with the Imhoff tank.

The Imhoff tank affords a method of obtaining separate sludge digestion in a chamber placed under the sedimentation chamber. It eliminates the necessity of transferring the settled matter from one tank to another, and thus has an advantage over the separate tank system. That it has some unpleasant tendencies, however, must be admitted.

In the experimental work in Brooklyn, three Imhoff tanks were used, designated, respectively, as No. 1, No. 2, and No. 3. The capacities per day were as follows: No. 1, 150 000 gal.; No. 2, 100 000 gal.; and No. 3, 50 000 gal., each operating on a theoretic 2-hour retention period. The tanks were of different depths, but of the same plan, the cross-section at any proportional depth being identical in all of them. All the tanks were operated more than five years.

Among the points of interest observed during the Brooklyn experiments, the speaker will mention the following:

1.—For Imhoff tanks the sludge digestion capacity should be sufficient to carry the sludge for 8 months without discharging, and the capacity below the slots should be at least about 2 cu. ft. per capita for the population served. This large capacity not only is necessary for storage during non-drying periods, but it also appears to have some important relation to the digestive process, and helps to prevent foaming.

2.—Double slots, that is, one slot at the bottom of each inclined plane of the sedimentation chamber, with a triangular baffle beneath, instead of one plane passing under the other, appear to give the best results in these tanks and largely eliminate the need of squeegeeing the slopes.

3.—The slopes should be at least 1¼ vertical to 1 horizontal, and the flow should be parallel with the slopes and slots; a rate of flow equal to 1 ft. per min. is excellent.

4.—Scumboards and baffles add to tank efficiency.

5.—From observation, the speaker has reached the conclusion that two sludge digestion pockets under the sedimentation chambers are better than three, and that the best length for the sedimentation chamber would afford 60 ft. net for the flow between the scumboards at the inlet and the outlet. Provision should be made to reverse the flow.

During the World War, and while in the employ of the U. S. Government, among various assignments, the speaker was detailed to take care of the design of an Imhoff tank plant for Harriman Village and other industrial housing projects, and the City of Bristol, Pa., jointly. W. H. Boardman, Assoc. M. Am. Soc. C. E., of Philadelphia, Pa., was retained by the City of Bristol, and collaborated with the speaker on this work, which consisted of three Imhoff tank units, each of which had a capacity to care for 2 500 000 gal. per day of sewage flow, at a retention period of 1 hour, or 1 125 000 gal. per day at a period of 2 hours. The principles previously mentioned were carried out in the design of the plant, which has been in operation about three years and has given excellent service. There has been no trouble with the plant, no odors, and squeegeeing of the slopes is seldom required.

Many engineers have come to the conclusion that the Imhoff tank is an element of risk in a treatment plant. After having visited most of the larger and many of the smaller installations of this tank in the United States and abroad, the speaker's opinion is that to a considerable extent its bad name is due to faulty design, and, still more, to faulty operation.

Relative to the retention period in tanks: Experience and experimental work indicate that this period should be short, usually not more than 2 hours, even 1 hour will give the best results in many cases. Sedimentation is very rapid during the first hour and septic action may intervene during the second hour, and it is always desirable to keep the liquid part of the sewage as fresh as possible. Experimental results as well as observations tend to show that although not affording the high percentage of removal obtained by longer periods, a period of 2 hours gives all the preparation required for sprinkling filters or for direct discharge into a waterway affording sufficient dilution.

As tanks of all kinds have always given some trouble, even if such trouble is merely psychological, which must be taken into account, a method of mechanical removal of suspensa from sewage has long been sought. The fine screen is the principal result of this search and the advantages secured by apparatus of this type are many and obvious. Fine screens act on the sewage at once as it enters the plant, preserving its freshness, and do not foul the night flow. They may be used as the sole method of treatment where the effluent is discharged into a waterway which at all times has sufficient volume to provide the necessary dissolved oxygen to receive it. The same remark applies also to tank effluents of all kinds. Neither the fine screen nor the tank provides a complete treatment, and, in many cases, the use of either method must be followed by filtration. The tank will remove more suspensa from the sewage than the screen.

The problem of sewage treatment is mainly the separation of solids from liquids. Ordinarily, fine screens may be said to remove from 12% to about 30%—even higher in some cases—of the settleable solids. This quantity is considerably less than is removed by the tank, but the greater freshness of the effluent from the screen, and its freedom from probable nuisance in most cases, more than offsets the difference. Either tanks or fine screens will prepare sewage satisfactorily for application to sprinkling and other filters, as was shown by the Brooklyn experiments; therefore, either can be constructed as the advance guard of a future filtration plant.

There are many fine screen plants in operation in the United States, which are giving satisfaction as the sole method of treatment. One such plant may be seen at the foot of Dyckman Street in New York City, and there is another of the same type in Brooklyn, on which observations and experiments extending over four years have been made. More such screen plants are proposed and will probably be soon built, as they appear to give all the treatment conditions require for the present and for a long time in the future.

Attention should be called to the need of providing grit catchers and coarse screens ahead of the treatment plants; also, to the need of tide-gates and submerged outfalls, where necessary.

Other forms of tanks, not mentioned, are not at this time of importance in this connection. The old septic tank still has its use in some places, and is well known to the sanitarian as a friend or as a foe. Some special forms of tanks are of interest, such as the Travis tank, the Kreamer tank, the Dortmund tank, etc., but the results of their use have not proved them the best for stream protection, and none of them has met with much success under American conditions. The various forms of chemical precipitation tanks are being abandoned everywhere as unsatisfactory.

Where an effluent from a tank, or from a screen, is good enough under local conditions to be discharged into a waterway, it is important that it should be as fresh as possible, in order to insure against local nuisance, and that it should contain a minimum of bank-forming material which usually can be removed almost completely by a tank or with a screen of proper fineness.

The finely divided flocculent matter and colloid particles in suspension will make considerable inroads on the dissolved oxygen present, but if this is of sufficient volume and the floc remains in suspension, little, if any, harm need be anticipated. The solution of the problem depends on the oxygen demand created by the biological food supply present, and, in every case, local conditions should be studied by a competent biologist before the plant is designed.

Although separate sludge digestion tanks are now coming into considerable prominence, thus far there has not been much uniformity in their design, and much study and experiment are desirable to develop this form of tank. In some cases, duplicate settling tanks have been provided, as in some of the Doten tanks, which are allowed to fill with sediment and sludge to the point of interference, and are then laid off for digestion, the other tank being used for sedimentation; and this method has met with success. Alvord has perfected a very interesting combination of settling and digesting tanks, but the field for improvement is still very wide in regard to this part of the subject.

In closing, attention may be drawn to the fact that many large cities which are not yet treating their sewage, are situated on waterways which for many years would afford ample supplies for disposal by dilution of a tank or fine screen effluent without further treatment, but which if contamination is permitted to go forward, will soon require far more costly forms of sewage treatment.

THE POLLUTION OF TIDAL HARBORS BY SEWAGE WITH ESPECIAL REFERENCE TO NEW YORK HARBOR

By Kenneth Allen,* M. Am. Soc. C. E.

Whether the waters are tidal or those of inland lakes, the fundamental considerations regarding harbor pollution are the same, but the effects are accentuated in sea-coast harbors for several reasons:

1.—The oscillations of the tidal flow alternate with periods of slack-water when solids settle rapidly to the bottom and form sludge banks.

2.—The presence of salt in the water tends to precipitate the soaps in the sewage, making the effluent noticeable by its milky appearance and promoting further sedimentation.

3.—Owing to the greater specific gravity of sea water, the warmer sewage rises rapidly to the top and spreads out in a thin layer, diminishing to a film of sleek which covers a large and readily distinguished area on the surface.

4.—As sludge deposits decompose, sulphureted hydrogen is formed in greater abundance in salt water than in fresh water, owing to the breaking down of the sulphates contained in the sea water. Sulphureted hydrogen is known as the most characteristic of the offensive odors of putrefaction.

In experiments made for the Metropolitan Sewerage Commission, the effect of salinity on buoyancy was well shown by releasing varnished croquet balls, weighted to a specific gravity of 1.000, at a given depth and noting the time of ascent. In a mixture containing 28% of sea water (specific gravity 1.007), the upward velocity was 3 in. per sec., while with a mixture containing 86% of sea water (specific gravity, 1.0215), this velocity was doubled.

The ascent of sewage from a submerged outlet would always be less than that of a solid ball, due to rapid diffusion forming a mixture constantly approximating in character the water of the stream. Therefore, the velocities mentioned may be taken as indicating the limiting maxima for sewage discharged in still water.

For the reasons stated, the discharge of sewage into salt water is very likely to be objectionable, particularly if it is discharged in a septic state, and, under otherwise similar conditions, more care is required for its proper disposal than in fresh water.

An excellent illustration is found in the discharge of nearly 100 000 000 gal. per day of septic sewage within a period of about 2 hours, about high tide, at Moon Island in Boston Harbor. This spreads rapidly over the surface so that several hundred acres of water have the appearance of undiluted sewage, although samples taken by the Metropolitan Sewerage Commission in 1911 indicated an abundance of dissolved oxygen at a depth of 1 ft. or more. In fresh water, diffusion would be much more general.

In the Borough of Manhattan, which to many is synonymous with New York City, the untreated sewage passes from about 180 outlets directly to the salt water of the surrounding streams. The result is that deposits form in the slips and under the piers so that dredging is necessary, and conditions offensive to the eye and nose result. This is also true of much of the Brooklyn and Bronx

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water-fronts, but conditions are better in the other less densely populated boroughs.

These unfortunate results of the present method of disposal could be largely avoided by the general introduction of fine screening, which is already being done at Dyckman Street, Manhattan, Hendrix Street, Brooklyn, and at the 43d Street, Oak Street, and Thirty-second Avenue outlets in the Borough of Queens. There would still be left in the water, however, a large part of the impurities in solution or in a finely divided state, making a continuous demand on the dissolved oxygen.

From 1909 to 1914, the depletion thus caused has been determined by the Metropolitan Sewerage Commission, Dr. G. A. Soper, M. Am. Soc. C. E., President, and, since then, by the Board of Estimate and Apportionment, New York City, Nelson P. Lewis, M. Am. Soc. C. E., and, more recently, Arthur S. Tuttle, M. Am. Soc. C. E., Chief Engineer.

The warm-weather record is interesting, as saturations are always high in winter. The most salient features are:

1.—The annual occurrence of total depletion for short periods in the Harlem River.

2.—The rapid decline in saturation in the Lower East River, reaching zero for the first time in September, 1921.

3.—The continued and general lowering of saturation in all the other main branches of the harbor, particularly at The Narrows, where a resultant of the depletion that has taken place in the Bay and rivers above may be noted.

It has been supposed that, although odors are not evolved as long as any supply of oxygen remains, they are likely to occur locally when channel saturations are less than 20 or 30 per cent.

Investigations have failed to support this view, as far as New York City is concerned, for during the past season a large part of the harbor has held less than these percentages of oxygen, without any resulting nuisance. Fish life, however, is undoubtedly interfered with, since it has been sufficiently well demonstrated that most edible varieties will not thrive where the percentage of saturation is under 30. The remedy for this phase of pollution lies in some form of tank treatment, by which most of the fine solids and some of the colloidal matter may be removed.

Large areas of the harbor covered with a film of oil may be mentioned as having caused much complaint recently. This condition has been prevalent along the Staten Island beaches and in the Lower East River, where it frequently extends practically from shore to shore. The cause is due in part to extensive oil works on Newtown Creek and at Bayonne, N. J., but more especially by the discharge of bilge water from incoming oil-burning steamers. It has injured the bathing beaches, increased the fire hazards, and interfered with fish life.

To summarize: The City of New York is discharging about 750 000 000 gal. of sewage daily into the harbor, mostly without any treatment. With the large tidal inflow of 12 700 000 000 cu. ft. of sea water at The Narrows and 4 700 000 000 cu. ft. from Long Island Sound twice a day, added to an average flow of upland water from the Hudson River of about 2 070 000 000 cu. ft. per

day, conditions such as obtained formerly at London, Glasgow, Naples, or Havana are not likely to occur soon here; but the rapid reduction of dissolved oxygen in the harbor is a danger signal which should not go unheeded.

Several important steps have been taken already to improve conditions: A number of intercepting sewers are, or soon will be, under contract, and within two or three years a number of modern fine-screening plants will be in operation. It is also hoped that a way will be found to provide a more thorough method of treatment for some of the more important outlets which are responsible for the polluted condition of the East and Harlem Rivers.

TREATMENT OF STORM-WATER

By John F. Skinner*, M. Am. Soc. C. E.

The sources of stream pollution commonly considered have been sewage and trade wastes. The discharge from storm-water sewers and from storm overflows on combined sewers is another element which in some cases may merit attention. Especially is this true when the stream is small, when it flows through built-up territory or a park, or when it is used for fishing or bathing, or the water is used for drinking by cattle or by man.

It has generally been considered that before sewage enters a stream it should be treated to remove:

(*a*) Heavy solids, grit;
(*b*) Floating and coarse suspended solids, screenings;
(*c*) One-half or more of the finer suspended solids, clarification;

and if the stream is small and sluggish so that insufficient dilution occurs:

(*d*) Oxidation of the effluent,

in order to avoid nuisance.

Economy usually dictates that the sewage treatment plant on a combined system shall receive only a part of the storm flow, while the remainder is discharged raw into the stream either at the plant or, as directly as possible, at convenient points along the main intercepting sewer, on the theory that, in time of storm flow, the dilution will be ample.

This is doubtless true if the stream is swollen at the time, for either the dilution is so great or such a load of other solids is washed in from the water-shed that the accession from the sewer is inappreciable.

However, if the storm-water discharge is due to a local shower, and the run-off from the water-shed of the stream is not generally augmented, the dilution may be reduced and the velocity may be so small that deposits will occur, the heavy material forming bars and the floating portions becoming entangled in the vegetation and littering the shores.

After a period of drought, when a sudden shower causes storm-water to debouch into the stream, the first flush carries the accumulated deposits from the sewers, accompanied by street refuse, followed by a greater dilution of rain water.

The discharge of storm-water being occasional, the finer suspended solids often may be oxidized without serious offense, or be carried on by the current if they are not entangled in other grosser deposit-forming materials. The treatment indicated for storm-water, therefore, is the removal of this coarse and heavy detritus before the effluent is admitted to the stream.

The heavy solids consist of grit, ashes, sand, gravel, bones, fruit pits, tin cans, rubber, leather, and objects in part made of metal, which deposits will settle in a current of 1 ft. per sec. If the larger objects are excluded by racks, the resulting deposit will be mainly grit, ashes, sand, and gravel.

The floating solids consist of wood, straw, leaves, fruit rinds, remains of fireworks, rubber, baseballs, and toys lost in the surface sewers. If racks

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are provided they will accumulate, in addition to the last mentioned, a collection of the coarse heavy objects mentioned previously.

The material to be removed, therefore, may be classified as grit and screenings. Coarse racks with bars from 3 to 4 in. apart will generally precede the grit-chambers and the finer screenings will be removed later.

If the proposed detritus plant is at or near a sewage disposal plant where attendance is constant, racks may be raked at any time and will occupy the least space if placed vertical or slightly inclined. If the construction is in an isolated location where only periodic attendance is contemplated, the racks should be set on a flat incline, 3 or 4 horizontal to 1 vertical, and a large area should be provided so that the accumulation from a single discharge will not completely blind the rack and form a dam.

A velocity of about 1 ft. per sec. for a period of from 1 to 2 min. is the best for removing the grit. Heavy material settled in this way should not average more than 5% organic matter and, ordinarily, will not be offensive.

Grit-chambers, 10 or 12 ft. wide arranged in parallel channels, will be convenient and, if desired, may be operated automatically by floats designed to cut in or out a sufficient number of channels to care for the flow at approximately the desired velocity. The grit may be excavated by a clam-shell bucket, removed in trucks or cars, and used for filling.

The finer screenings may be handled in various ways. When at or near a sewage disposal plant, mechanical screens may be used. At the lower end of a grit-chamber, 10 or 12 ft. wide, carrying about 40 000 000 gal. per day, a 12-ft. R.-W. screen, or a 12-ft. Dorrco screen 10 ft. long, may be used if properly housed and provided with power.

Another method would be to construct the grit-chamber or basin with the upper part of the side next to the stream of open permeable material, so that water may pass through it leaving the screenings deposited on its surface. Maintaining a proper velocity through the chamber and keeping the porous bank from clogging would be the chief difficulties with this design.

Another suggestion which has considerable promise, is a grit-chamber preceded by coarse racks and provided at intervals of about 25 ft. with several sets of inclined scum-boards continued downward with inclined racks, progressively finer down stream, each of which is provided with raking platforms above and short submerged platforms projecting up stream from their lower ends to retain the screenings. These racks will not reach the bottom of the chamber, but will stop about 4 ft. above it. The down-stream end of the chamber will consist of an overflow weir which will maintain the water in the channel at the desired elevation. A drain valve for re-watering will also be provided.

All such devices will require periodic attention and cleaning and cannot be operated satisfactorily, except on a methodical schedule directed by a responsible man.

As previously mentioned, structures for treating storm-water may be made up conveniently of units about 10 or 12 ft. wide by a maximum of 10 ft. deep and from 60 ft. to 120 ft. long, designed for a velocity of 1 ft. per sec. The number of such units will depend on the quantity of storm-water antici-

pated, assuming the capacity of each channel to be 60 cu. ft. per sec. Either there must be a sufficient number of such units to carry the estimated maximum flow, or a smaller number may be constructed and a by-pass provided, as economy may dictate.

The simplest way of accommodating an extreme flow would be to construct the walls of the channels with considerable free-board, in order that an increased head on the outlet weir would accommodate the increased discharge.

As an example of about what would be required, let us assume a water-shed of 1 000 acres, 1 mile wide, and nearly 2 miles long, with the longest sewer 12 000 ft. in length from source to outfall, and with grades such that, when running full, the velocity will average 5 ft. per sec. It will require, therefore, 40 min. for water to travel down the entire system, and if a period of 5 min. is allowed for the water to reach the sewer, a shower of 45-min. duration will be the shortest in which the entire water-shed will contribute and precipitation from the remotest corner of the territory reach the outfall at the same time as from every other part. If from an examination of the rainfall records of the locality, one selects the 45-min. shower of such intensity that it occurs every year or two, it may be used as a basis for the computation of the run-off to be provided for at the outfall.

In Rochester, N. Y., this 45-min. shower has a rate of 1.22 in. per hour. In the residential section about 25% of the precipitation from these violent showers reaches the sewers during the period. A discharge of $1\,000 \times 1.22 \times 0.25 = 305$ cu. ft. per sec., therefore, can be computed, which will require five of the grit-chamber units mentioned. Such a plant will take the discharge of a 9-ft. sewer at a grade of 1 in 1 000, or a 7-ft. sewer at a grade of 1 in 280.

This may be roughly stated as three of the previously mentioned detritus chamber units per square mile of tributary territory, subject, however, to wide variation, depending on the rainfall, slope, and character of the territory. A similar approximation would place the cost of construction of such a detritus plant at from $10 000 to $12 000 per square mile of territory.

It is appreciated that little has been done in storm-water treatment and that the health authorities have been interested chiefly in the protection of streams from other pollution, but the time may be anticipated when the concentrated storm-water discharge of cities will demand attention.

POLICIES OF THE ENGINEERING DIVISION OF THE PENNSYLVANIA DEPARTMENT OF HEALTH AS TO PUBLIC SEWERAGE

By W. L. Stevenson,* M. Am. Soc. C. E.

In 1905, the Legislature of Pennsylvania enacted the "Purity of Waters Act" for the protection of the public health by controlling public water supplies and by prohibiting the discharge of sewage into the waters of the State.

The Act, however, provides that, in the case of municipally owned sewers, the Commissioner of Health, with the unanimous agreement of the Governor and the Attorney General, may permit the discharge of sewage subject to such conditions as he deems will subserve the general interests of the public health.

The evident intention of this prohibition and provision for discharge under certain conditions is to provide for varying degrees of sewage treatment, so as to maintain the streams in a reasonably clean condition and to make it practically and economically possible to use certain waters of the State as sources of public water supply and for other purposes requiring a hygienic standard.

Streams are the natural drainage channels for rain water flowing over the surface of the land and conveyed to them by storm-water conduits from the towns on their water-sheds.

It follows, therefore, even if all the sewage was excluded or completely purified, that no surface water from a populated water-shed would be fit for use as a public water supply without some form of purification to eliminate the natural contamination of the surface stream.

It is possible to produce a clean, colorless, and bacteriologically safe water supply from a grossly polluted and contaminated source. It is also possible to purify sewage to such an extent that it will be freed from all its polluting and contaminating constituents; but both procedures are exceedingly costly and often unreliable. Therefore, a wise and just administration of the "Purity of Waters Act" should be based on an economic consideration of the uses and conditions of the various waters of the State so as to secure the greatest protection to the public health by the least expenditure of public and private funds for the construction and operation of water purification and sewage treatment works.

The Engineering Division of the Pennsylvania Department of Health is charged, among other things, with the examination of plans of sewerage and sewage treatment works, submitted with applications for issuance of permits; also, with making field investigations and recommending to the Commissioner of Health the conditions under which permits may be issued.

Certain fundamental policies have been established to obtain as nearly as possible uniform practice in this work, but it must be borne in mind that the wide range of uses and conditions of the waters of the State makes it

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utterly impracticable to follow detailed policies that will be State-wide in application.

In this discussion, the word, "stream", is considered as including all lakes, ponds, or other waters of the State, and where policies are given for streams used as sources of public water supplies, they apply equally to streams used for other purposes requiring a hygienic standard, such as bathing and ice harvesting.

Classification of Streams.—The problem of administering that part of the "Purity of Waters Act" relative to sewerage has three phases: (1) prevention or abatement of nuisance; (2) protection of sources of public water supply; and (3) maintenance of clean streams.

Even though they are not used as sources of public water supply, streams as a matter of decency should be maintained in a reasonably clean condition, that is, free from sludge deposits on the bed or banks, ebullition of offensive gases, undue turbidity, discoloration, floating solids, or any other nuisance to sight or smell of sewage origin.

In addition to being reasonably clean, streams used as sources of public water supply should provide a raw water sufficiently low in organic and pathogenic bacterial content that it can be purified safely and economically for domestic purposes.

Small streams flowing through highly developed suburban territories or land devoted to park purposes for the pleasure and recreation of the public, demand a high degree of protection and should be so clean that they show no appreciable evidence of sewage.

Degree of Sewage Treatment Required.—The ground-water drained or pumped from coal mines contains considerable sulphuric acid which acts as a germicide on sewage discharged into streams receiving such mine drainage, thus minimizing the danger of creation of nuisance or menace to the public health; also, these streams frequently carry large quantities of fine coal which mask any visible evidence of sewage matters. Therefore, at points reasonably remote from water-works intakes, sewage generally may be discharged untreated into these streams, but preferably from the outlets of the several drainage areas and always into the main current of the stream, in order to prevent local nuisance at the immediate point of discharge and to distribute the sewage more effectively in the stream and thus take full advantage of dilution by the water of the stream and the germicidal action of the acidity.

If the rate of flow and velocity of streams not used as sources of public water supplies are insufficient to assimilate crude sewage inoffensively, then such a degree of treatment should be required as will lighten the load of organic matter on the stream so that the desired conditions of cleanliness will be maintained, even during times of drought.

The assimilating power of the stream should be distributed equitably among the various municipalities along its course, so that one town will not be required to treat its sewage to a high degree because a neighboring community up stream has overtaxed the stream with insufficiently treated sewage.

In the case of streams used as sources of public water supplies, sewage treatment should be required to a sufficient degree to produce an effluent which, after being carried by the stream to the water-works intake below, will not prejudice the safe and economical purification of the water for domestic purposes.

Ample diluting water and sufficient distance between the water-works intake and the place of sewage discharge may reduce the required treatment of sewage to a negligible degree.

Cleaning Streams.—The cleaning of sewage-polluted streams not used as sources of public water supplies or the maintaining of such streams in a clean condition should begin at the upper end and progress down stream.

In general, action taken to protect streams used as sources of public water supplies, subject to local conditions, should begin at the first source of sewage contamination above the water-works intake and progress up stream.

Sewer System.—Every municipality should prepare and submit to the State Department of Health for approval, a comprehensive plan of sewerage before undertaking the design of new sewers or the extension of existing sewers, as may be needed from time to time.

In sewering new drainage areas, comparative estimates of cost should be made of:

1.—Collecting sewage and rain water in separate conduits; and
2.—The use of the combined system with interception of all dry-weather flow of sewage and such a percentage of storm water contaminated with sewage as will afford the required degree of protection to the receiving body of water.

In extending existing combined sewers in municipalities having well improved highways, the relative economy of continuing the combined sewers, or of replacing them with separate sewers, should receive careful consideration in connection with the relative protection afforded to the waters of the State.

Contiguous or adjacent municipalities should be urged to give careful study to the possible advantage of entering into agreements for the joint construction and operation of intercepting sewers and sewage treatment works.

Sewage Treatment Works.—In cases where sewage treatment is not required at present, general outline plans of the project should be submitted to the State Department of Health for approval. They need only be prepared in sufficient detail to demonstrate the feasibility of constructing the intercepting sewers and sewage treatment works and to show that the site of the proposed work is properly located, ample in area, and of suitable topography for the erection of treatment works sufficient for the needs.

The advance in the knowledge of sewage treatment in past years indicates the possibility of even greater improvements in the future, and, therefore, it is inadvisable to prepare detail plans until it becomes necessary to construct the works.

If the approved site is likely to be acquired for private use, the municipality should forthwith acquire it, so as to be prepared to treat the sewage when necessary.

In cases where the use and condition of the stream is such that sewage treatment is required, the plans should show in detail that part of the works required for the needs of the present and reasonable future and, in general, outline the extensions required for the increased rate of flow and higher degree of treatment that may be needed in the more distant future.

The operation of oxidizing processes of sewage treatment works, the only purpose of which is to prevent the creation of a nuisance in a stream not used as a source of public water supply, may, on request and with the specific approval of the State Department of Health, be suspended during the colder part of the year, when the greater quantity of dissolved ogygen and lessened biological activity in the stream will permit the discharge of unoxidized sewage.

STREAM POLLUTION AND ITS CONTROL

By Earle B. Phelps,* Affiliate, Am. Soc. C. E.

A complete statement of the problem of the control of stream pollution, aside from its purely legislative and administrative aspects, involves three essential terms capable of expression in common units. These are: The capacity of the stream; existing or contemplated pollution load; and, means of reducing the pollution load.

Every stream has a certain maximum capacity for the biological disposal of common organic wastes. According as local circumstances permit a greater or less depletion of this natural reserve of capacity, with consequent temporary reduction in stream quality, any stream may be said to have an effective working capacity for waste disposal which is a function of the fixed maximum capacity and of the permissible quality depletion. The latter varies according to circumstances and usage, but the lower the standard of minimum acceptable quality the greater the working capacity of the stream for waste disposal.

This working capacity is not a mere capacity for dilution. It comprises all those bio-chemical reactions which make for the self-purification at the stream. A pollution load of stated amount brings about, within a certain distance (the critical point, measured down stream in hours), a definite reduction in stream quality, after which the operation of self-purification becomes predominant and stream quality rises again toward normal. A greater load produces a proportionately greater reduction in quality at the critical point, and subsequent recovery is proportionately delayed.

The maximum load previously referred to, may be defined as that load which, at the critical point, produces conditions inimical to those agencies which bring about self-purification, so that improvement of the stream beyond that point is either greatly delayed or altogether lacking.

Although the available data in this field of investigation are too meager to justify any definite generalization for all streams, yet the methods of investigation have been developed sufficiently at the present time to permit the determination of the fundamental stream constants, the critical point, the maximum capacity, and the rate of self-purification in the case of any stream of known physical characteristics and pollution load. By the aid of these constants, the effect of added or diminished pollution load can be determined and, conversely, for any fixed lower limit of quality, the working capacity of the stream becomes known. This information is the first prerequisite in any comprehensive program for the control of stream pollution.

A more detailed account of the various subdivisions of this phase of the problem is not possible in this discussion, further than to indicate that it is not simple, but is essentially dependent on sedimentation, additional pollution or dilution in the lower stretches, and re-aeration; and that the latter, the most significant factor of all, is a function of depth, velocity, turbulence, and other physical conditions.

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It must further be pointed out that all references to stream quality must be in terms of a unit which may also be used in measuring the pollution load and the effect of remedial measures, the so-called oxygen unit, to which reference will be made later.

The second term in the stream-pollution formula is the pollution load. This phase also is complicated in practice and may be dealt with only in general terms in the present summary. Obviously, the oxidation of organic matter in the stream is a time function so that a first approximation to a statement of pollution load must be in terms of total oxidizability and rate of oxidation at each degree of completion. Given this complete oxidation curve, at any stated load, the effect on a stream of known capacity could be determined. Fortunately, the form of this curve has been established so that two determinations on the oxidizability of a waste, taken, say, at 24 and at 48 hours, furnish the necessary and sufficient data. This treatment assumes, however, that all the pollution travels uniformly with the stream up to the point of complete disposal. If a part of the waste is capable of settling and the stream characteristics permit of sedimentation, the oxidation characteristics of the settleable and the non-settleable parts must be dealt with separately with appropriate time factors.

The methods by which the pollution load is measured in the laboratory have served to indicate the only satisfactory unit of measurement for stream quality. The earlier work in this field attempted to deal with pollution in terms of nitrogen and to establish some empirical relation between the content of unoxidized nitrogen in the stream and that elusive factor, stream quality, for which there was no adequate direct measure. These attempts were theoretically unsound and led only to greater difficulties and to such fallacies as the establishment of a constant dilution ratio for sewage in a stream, which was supposed to mark the boundary between safe and unsafe practice. The newer chemistry deals wholly with oxygen units, as these units measure pollution load as well as stream quality and are interchangeable between the two when once the fundamental formulas have been derived. The characteristics of the most important polluting agent, domestic sewage, have been quite satisfactorily determined and are referable to the population base, thus eliminating the former difficulty of determining the strength and total volume of the sewage, neither of which *per se* has any but incidental relation to the question of pollution load. Much information has also been accumulated concerning the characteristics of certain of the more important industrial wastes, such as the waste from tanneries, soap works, and others. Here, too, the attempt has been made to refer the pollution load to the reference base, manufactured product. It is obvious that there must be some fairly constant amount of pollution resulting from the cleansing and tanning of a single hide by a standard procedure, regardless of the quantity of water in which the waste is carried to the stream.

With the determination of the stream constants and of the present and prospective pollution loads, the condition of the stream resulting from any assumed increase or decrease in the load may be stated with reasonable approximation. If this condition requires improvement, the degree of

improvement decided on is also capable of expression in terms of the pollution load. The problem then becomes one of treatment of the polluting wastes to the required degree. Except in certain fields, such as the treatment of domestic sewage, this leads to experimental studies of remedial measures which are essentially chemical or biological in nature, rather than engineering. Domestic sewage has been sufficiently investigated so that the engineer now has at his disposal the necessary bio-chemical data to enable him to design works for any specific degree of reduction of the pollution load. A proper appreciation of the basic principles of stream pollution, however, will enable him to design scientifically for the utilization as well as for the protection of the stream, a matter which has been too often overlooked. A considerable field of activity in the remedial treatment of industrial wastes is still open to investigation. Many of the wastes do not lend themselves to the line of procedure suggested by the speaker. The effects of some wastes, such as saw-mill refuse, are purely physical, while the effects of others are purely chemical; the acid residues from certain iron works are an example of the latter. In the majority of cases, however, the effect is bio-chemical, and the results of experimental studies are best stated in terms of oxygen demand pollution units which can be compared directly with the stream capacity similarly stated.

Thus, a scientific basis is laid for the study of this problem of stream conservation which, properly understood, means the maximum utilization of stream resources. In addition to the three scientific aspects of the problem, there are the legislative and administrative aspects without mention of which this discussion would be incomplete. The matter has been placed last, however, because it should rest wholly on a correct understanding of the fundamentals. Much harm has been done to real progress by inverting this natural order and through misdirected legislation, seeking an impossible perfection which has only invited disrespect and lack of enforcement. The successful stream-pollution legislation of the future will give wide discretionary power to capable administrative bodies whose duty it will be to determine stream resources and capacities and to distribute these so that a maximum utilization will be made possible. The interstate character of most of the great streams of the United States, and the magnitude of the problem in both its investigative and its administrative phases suggest Federal control as a probable final outcome. This is emphasized more particularly by the present failure of most of the States to deal with the problem on any comprehensive basis.

The matter of stream pollution and its control is now essentially one for the health authorities. Historically, it has had conspicuous public health aspects, but time has been lost and attention misdirected by too close adherence to this traditional treatment. The problem of stream pollution is clearly one of conservation of a natural resource with, at times, a public health aspect which is always paramount when present, and, at other times, an overshadowing commercial and industrial aspect. In short, it is a problem in the maximum utilization of an economic resource and its future development should be placed in the hands of an authority having the widest possible scope and the broadest jurisdiction.

DEPOSITION OF SLUDGES RESULTING FROM SEWAGE DISPOSAL PLANTS

By T. Chalkley Hatton,* M. Am. Soc. C. E.

The problem of sewage disposal will be presented herein from a different angle than that usually discussed in papers on this subject, and the speaker would like to emphasize a certain viewpoint.

Sewage disposal embraces two distinct problems, the partial purification of the liquor and the final disposition of the resultant sludge. Up to the present time the great bulk of scientific labor has been devoted to solving the first problem, the solution of the latter having been left largely to chance.

The speaker believes that the reason for this is principally economic; that the nuisance to our neighbor using a stream polluted by our sewage demands its partial purification, whereas the disposition of the sludge is more likely to be a nuisance from which we alone may suffer.

Science has discovered several processes of purifying the liquor to any standard which may be required to prevent a nuisance, but the speaker is not so sure that it has been as successful in disposing of the sludge, and he is firmly convinced after many years of intensive study of this phase that sanitary engineers should give it far more consideration than they have.

Engineers cannot expect to make sewage disposal popular with the public if about the disposal plants are spread foul smelling, dirty looking, putrefactive piles of material that breeds flies and vermin which the surrounding population believe infect their properties.

A casual inspection of the sewage disposal plants of England when the sludge is being disposed of on property adjacent to the works convinces the engineer that the work is being only half done, and from personal inspection in 1907, and again in the early part of 1921, the speaker can state positively that, as far as he was able to judge, no improvements worthy of note had been made in the 14 years, and that the nuisance had so multiplied as to threaten the welfare and health of large numbers of people.

Take the experience on this Continent and consider the nuisance being maintained from sludge disposal at Toronto, Ont., Canada, and at Baltimore, Md., two of the largest cities on the Continent which are operating modern sewage disposal plants.

In both these cities, suits have been brought and damages recovered for nuisances arising from sludge disposal. Mr. Marks, of the Pennsylvania State Board of Health, has stated to the speaker that the present methods of disposing of sludge in the sewage disposal plants in Pennsylvania were becoming an unmitigated nuisance and some other means would have to be devised.

There is not a sewage sludge produced from any sewage disposal process in which lime is not used, but that has some value as a fertilizer. Such sludge will be sought by the agriculturist if it can be delivered to him in a condition in which he can handle it conveniently, and the speaker believes that is the

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final solution of the problem. It will cost far more in some, in fact, in many, instances to procure this condition than the returns from the sale of the sludge; in fact, there may be instances when no monetary returns whatever will be secured.

Whether it does or does not return a monetary value directly, it does get rid of the filth of the human body in the only logical way by returning it to the soil from whence it came, and indirectly will pay just as the purification of water supplies has paid.

Engineers have had the direct economic viewpoint too much in mind to study this sludge problem properly, and as municipal engineers charged with improving the sanitation and welfare of the communities they must look on it with a broader vision or sewage disposal as a science will never attain the place it deserves.

The viewpoint of the German engineer has been different from that of the English and the American engineer. He has not been as anxious to produce a stable effluent as he has to get rid of the sludge in a way which may add to the wealth of his community, and one who goes about the German sewage disposal plants to-day will be struck with the entire absence of piles of sewage sludges. If one wants to see them, he must travel about the neighboring farms where they will be found placed in compost beds to be used to fertilize the soil as the season demands.

The speaker desires to give just one illustration of what can be done by a little useful and intelligent propaganda among the farming community adjacent to a sewage disposal plant.

The city authorities of Rochester have built and operated one of the largest Imhoff tank systems of sewage disposal in the United States. It was designed to dry the sludge produced on beds and dump it into a beautiful valley adjacent to one of the fine parks belonging to the city. The engineer in charge of the works soon realized that this would finally result in a nuisance, and he went among the fruit growers near-by and induced them to try some of this partly dried sludge on the ground about the trees.

The next season these growers hauled away all the sludge they could and paid a small sum therefor. The demand became so great the following season that the price was increased about 50%, but this increase had no effect and, to-day, the sludge is being disposed of at about the same price which it costs to de-water and deliver it to the customer. It is believed that in another season or two the authorities will be able to dispose of all the sludge the plant will produce. What this engineer has done, other engineers can do, if they go about it in the right manner, but they must first get the vision that the sludge must be disposed of finally by putting it back on the ground regardless of the cost.

Sludges have been successfully de-watered in several places in America and in Germany. At Providence, R. I., and at Worcester, Mass., filter presses have been used for many years, while in Germany centrifuges have been used with equal success. In Baltimore, during the past five years, much of the sludge has been de-watered at 12 and 15% and sold as a fertilizer, and about 25 tons per day of dry material is thus being produced.

Whether or not the Baltimore experiment is self-supporting, the speaker cannot state, as he knows nothing of the cost of the overheads. The original contractor who undertook this work four or five years ago is still on the job, although his original contract was for two years only. How he continues with the present lack of demand for fertilizer is more than the speaker can tell, but he is continuing nevertheless. The contract thus far has been profitable to the city.

In Milwaukee, this sludge-disposal problem has been studied for five years, and although it has not been solved satisfactorily, the point has been reached where the moisture content has been reduced by sludge presses and dryers to 10%, and if the demand for fertilizer is restored to pre-war conditions, it will be possible to dispose of the sludge for at least what it costs to produce it and probably a profit will be enjoyed.

THE DILUTION FACTOR

By Langdon Pearse,* M. Am. Soc. C. E.

Discussion of stream pollution in the early days appears to have centered mainly on the standpoint of nuisance. As concentration of population has increased, together with the load on the stream (or other body of water), other problems have entered, such as the effect of the sewage and sludge on fish life, shellfish, and bathing. In more recent years, in many localities, consideration has been demanded for the degree of loading which can be handled by a water purification plant. From this gradual development, two widely distinct dilution factors may be said to have arisen, one relating to prevention of nuisance, the other due to prevention of undue load on the water purification plant. On the first of these factors, considerable data are available; on the second, comparatively little is known as yet. The purpose of this brief discussion is not so much to summarize existing data, as to point out the need for well considered investigations over considerable periods of time on stream pollution problems, and to cover quantitatively the subject of loads on water purification plants.

In the problem of stream pollution, the dilution factor of nuisance is of prime importance and has provoked discussion covering the past 35 years. Many authorities have endeavored to state this factor on a basis of cubic feet per second per 1 000 population. These bases have been largely varied by the type of city studied, and the character of the stream, as shown by results in Massachusetts and elsewhere. In general, a range of from 4 to 7 cu. ft. per sec. per 1 000 population has been indicated, with a lower limit of 2 and an upper limit of 10 cu. ft., according to the authority.

In England, high dilutions have been recommended by the Royal Commission, as noted by Mr. A. J. Martin, upward of 27.8 cu. ft. per sec. per 1 000 population having been mentioned.

In the problem of the Sanitary District of Chicago, an expression was early sought for a somewhat different angle of attack, namely, for the amount of oxygen required. For this determination, the bio-chemical oxygen demand test has seemed to be the most favorable. Continued tests on a large sewer serving 300 000 people, with no marked industrial wastes, have given an equivalent of 0.22 lb. of oxygen per capita for complete oxidation. This requirement is based on an incubation period of 10 days, in sealed bottles, and takes no account of re-aeration factors.

Inasmuch as the oxygen content of water varies with the temperature, for translation into more familiar equivalents, Table 1, based on the 0.22 lb. per capita, may be helpful. The figures given therein are indicative of what would be needed if no re-aeration was to occur. The re-aeration factor should reduce the flows required per 1 000 population. Of the oxygen requirement, roughly, about from 20 to 30% seems to be necessary in the first 24 hours.

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TABLE 1.

Dissolved oxygen, in parts per million.	Temperature corresponding to dissolved oxygen saturation, in degrees Fahrenheit.	Flow required in cubic feet per second per 1 000 population.
6	108	6.80
8	81	5.09
10	60	4.07
12	46	3.40
14	35	2.92

Present observations on the Main Channel at Chicago appear to indicate that a higher dilution than the minimum of 3.33 cu. ft. per sec. per 1 000 population is required to dilute the raw sewage of domestic origin, in the summer; between 4 and 5 cu. ft. per sec. now seems to be nearer the requirement. With an industrial load at times more than 50% of the human load, the need of much higher dilution for both domestic and industrial sewage combined is apparent. The exact figure will depend on re-aeration factors and seasonal conditions, such as temperature and the extent of the ice sheet, as well as the standard of dissolved oxygen to be maintained at critical points.

Data on re-aeration are needed. The extent will depend on the swiftness and turbulence of the stream. In traveling from Marseilles to Chillicothe, Ill., a distance of 65 miles, the Illinois River absorbed about 205 000 lb. of oxygen from the atmosphere, on one test, approximately 3.8 parts per million.

From the standpoint of sewage dilution, the problem is reduced to the determination of the amount of pollution to be handled and the amount of oxygen available plus re-aeration. To make a balance, additional dilution, or supplementary treatment, may be required. The exact solution will hinge also on the rate of oxidation and the amount of residual oxygen desired at critical points.

The amount of residual oxygen to be preserved will depend on the conditions to be met. Where nuisance alone is to be avoided, there seems to be little doubt but that some residual oxygen must be present. This may not prove to be sufficient to prevent odors from decomposing sludge banks.

From the standpoint of fish life, considerable oxygen is required. Although fish may live in low oxygen content for a short time, they cannot survive long. Fish specialists suggest not only the study of the oxygen content, but also the carbon dioxide and the hydrogen-iron concentration.

A number of years ago, Dr. Arthur Lederer and the writer made an extended investigation of the fish question for the Sanitary District of Chicago, with the following conclusions:

1.—Fish life is affected by material in suspension and solution, and the variations thereof.

2.—Material in suspension may cause disease by fungus growths or death by mechanical stoppage of the gills or suffocation thereby. The character, size, and quantity of the material must be considered. A careless and continuous handling of fish may induce fungus growth through the abrasion of the slimy, protective coating of the body, thus allowing foreign bodies to adhere.

3.—Material in solution includes mineral and organic matter and gases. The investigation covered only dissolved oxygen, which, of the gases in solution, was the most important in this case.

4.—For continuous fish life, a content of dissolved oxygen of at least 2 to 3 parts per million is required by practically all the fish used in the investigation.

5.—The fish studied may live for very short periods in a content of dissolved oxygen of between 1 and 2 parts per million. This provides time for the fish to escape to better conditions, if any are available.

6.—English observations, as well as German experiments, indicate that, in general, 25 parts per million of dissolved oxygen are about the minimum for fish life and that for continuous thriving fish life more is desirable. In the light of the experiments in America, it appears that many species will live in 25 parts per million, but that more is desirable.

7.—Sudden changes in the character of the environment are undesirable and may cause the death of the weaker specimens.

8.—Of the fish experimented on, their vitality or their relative ability to withstand low contents of dissolved oxygen, is roughly in the order designated as follows, the strongest being listed first, the weakest last: German carp (from Illinois River); catfish and bullhead (from Illinois River); black bass (from Illinois River); yellow perch (from Lake Michigan); and sunfish (from Lake Michigan). The pickerel and golden shiner apparently are to be classed with the sunfish. Further data, however, are desired, as the tests on these two species are not as extended as on the others named.

An extended joint investigation by the U. S. Public Health Service and Sanitary District of Chicago has been undertaken which will cover in detail the self-purification of the sewage of the District from Lake Michigan to the Mississippi. Many of the points suggested herein will then be elucidated. The effect of re-aeration, temperature, ice sheet, time or length of travel, and sludge travel, are to be studied, and existing data will be summarized. With the work already done by the Public Health Service on the Potomac River and the Ohio River, this third investigation should provide useful data for all students of stream pollution problems.

The dilution required to prevent an undue load on water purification plants appears to have been first cast in definite form in the work of the International Joint Commission. The standard suggested was that the average load on such a plant should be such that the raw water would not contain as a yearly average more than 500 *B. coli* per 100 cu. cm., or, as translated by Professor Phelps, a dilution of 4 cu. ft. per sec. per capita or greater should not produce an undue load.

In a paper entitled "The Loading of Filter Plants", H. W. Streeter, Assoc. M. Am. Soc. C. E., has discussed the problem in some detail from the standpoint of the efficiency of the filter plant, corroborating in general the report of the International Joint Commission. However, he suggests the need of more data and a definite adoption of a standard for filtered water.

Many filter plants delivering a reasonably safe potable water, judged by the typhoid fever death rate, handling raw waters more polluted than those mentioned. How safe they will be, experience alone will tell. The need at present is for careful standardized tests of such plants in order to determine the data along common lines. The U. S. Public Health Service has made an extended survey of the Ohio River Basin, and more studies might be made

available from scattered plants, by the more general adoptions of standard methods of analysis and report.

Such data will be helpful in solving the problems ahead, not only on the Great Lakes, but on many of the large rivers of the United States. Further, such data will be helpful in crystallizing the policy of various State board of health authorities who have jurisdiction over different parts of the same stream, as in the case of the Missouri River from Kansas City to St. Louis, Mo.

There is, at the present time, need of a sane, well-considered viewpoint, looking ahead, not only from the standpoint of nuisance, but from the standpoint of fish life, and in many cases of drinking water. Various organizations of sportsmen, nature lovers, conservationists, and others are urging the restoration of streams to their former condition of virgin purity. How far it is necessary or possible to go in each individual case is a question for the engineer to solve. For the solution, every available experience helps.

The writer desires to acknowledge the courtesy of the Trustees of the Sanitary District of Chicago and of Albert W. Dilling, Assoc. M. Am. Soc. C. E., Chief Engineer, for the use of certain data contained herein.

PREVENTION OF MISUSE OF SEWERS

By W. H. Dittoe,* M. Am. Soc. C. E.

The purpose of this discussion is to call attention to the abuse of sewerage and sewage disposal systems resulting from failure of municipal officials to control properly the establishment and use of connections to sewers. Sanitary engineers generally have deplored this condition of affairs and, as a class, are in agreement that better control should be provided. However, they have been prone to leave entirely to municipal officials the solution of this problem and with few exceptions have failed to recognize that it is essentially their duty to take the lead in establishing proper control. This discussion will attempt to show that sanitary engineers must not only recognize the importance of preventing abuse of sewerage and sewage disposal systems, but must undertake the problem of prevention if the expected efficiency of such improvements is to be realized. The speaker will not recite instances to demonstrate the evil effects of misuse of sewerage systems, with which all engineers are familiar, but will confine this discussion to general statements with the hope of arousing an interest on the part of engineers in attacking a solution of this problem.

One of the most important factors reducing the efficiency and value of systems of sewerage and sewage disposal is the misuse of sewers. Sufficient emphasis has not been placed on this subject, and rarely is it found that a municipality enforces a strict policy regarding the use of sewers. Considerable effort is expended by engineers in designing sewerage systems and sewage disposal works and, as a basis for such design, the volume and character of the sewage flow must be known or estimated. Therefore, if these factors are disturbed appreciably, the improvements will not be used under the conditions for which they were designed. The result will be a shorter life of the system as a whole, impairment of its efficiency, and generally unsatisfactory results. The misuse of sewers is also an important fault affecting the successful operation of sewage disposal works, and, therefore, the problem of prevention of stream pollution. It is useless to design sewerage improvements on an assumption that certain maximum rates of flow will occur and that the sewage will be of a certain character, unless the construction of such works is followed by the enforcement of a definite policy which will insure against exceeding such rates or changing such character.

Sewers are designed for definite purposes and when used for other purposes may be said to be misused. Storm drains are misused if they receive sewage, industrial wastes, or other waste of objectionable character. Combined sewers are properly used for the removal of practically all classes of liquid wastes, but are misused if they receive industrial wastes affecting the sewerage system or process of sewage treatment. Sanitary sewers, as the name implies, are for sanitary purposes only, and are misused if they receive drainage from the surface and roofs, subsoil drainage such as may be admitted by building foundation drains and through open or leaky joints, and industrial wastes of a character to affect the sewerage system or treatment process.

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The effect of the misuse of sewers is frequently quite serious. Sewage discharged into storm sewers causes nuisances at outlets and offensive odors through street inlets. If such practice is permitted, the benefits to be expected from a separate sewerage system are not realized. The admission of surface and subsoil drainage to sanitary sewers overtaxes the sewerage system, resulting in cellar flooding and damage to sewers, and overburdens the pumping equipment and sewage treatment works, necessitating by-passing of the flow, impairing the efficiency of the plant, and frequently causing rapid deterioration. Industrial wastes often cause clogging or may actually destroy the sewers if the wastes have solvent properties. Many industrial wastes also interfere seriously with the efficiency of sewage treatment plants. Of the more troublesome industrial wastes, may be mentioned: Wastes from tanneries and glue factories containing hair and lime; wastes from textile industries containing cloth, fibrous material, and objectionable compounds in solution; wastes from canning factories containing vegetable particles; wastes from stockyards containing manure; wastes from packing plants containing animal offal; gasoline wastes from garages; acid wastes from metal industries; wastes from milk industries; and gas-house wastes. Such wastes should be treated properly prior to their discharge into the public sewers, or they should be excluded entirely.

It seems apparent that the evils resulting from the misuse of sewers are of sufficient importance to warrant an effort to prevent it. It has been accepted by many engineers that the misuse particularly of sanitary sewers to carry storm water, is inevitable and cannot be prevented, and this conclusion has produced a strong argument for the selection of the combined system. It is true that the combined system can rarely be misused and that from this standpoint it is preferable; however, it does not appear sound to conclude that it is impracticable or impossible to secure the proper use of separate systems. In fact, separate systems now in existence will continue in use and new systems will be built; therefore, engineers and city officials cannot avoid the responsibility for securing their proper use. The industrial waste menace is present and must be controlled regardless of the sewer system used.

It is obvious that the proper use of sewers cannot be secured without strict enforcement of ordinances and regulations by the proper municipal officials. The sewer contractor and the property owner cannot be expected to realize the importance of using the sewers properly and for the purposes for which they were designed to function, and, therefore, they must be controlled in order that the public may not suffer from their mistakes. Ordinarily, municipal officials themselves do not appreciate the necessity of protecting sewers and sewage treatment plants from abuse, and, therefore, are not in a position to initiate suitable regulations. It seems apparent that it becomes the duty of sanitary engineers to dictate such regulations and to see that they are adopted. This function of the engineer is as important as the design and supervision of construction of the improvement, and if it is not performed it may truly be said that the work of the engineer has not been complete. Ordinarily, the engineer who has designed and supervised the construction of a sewerage system and a sewage treatment plant furnishes definite instruc-

tions in regard to the operation and maintenance of the system, and he should likewise furnish a definite program for preventing misuse which may defeat the purpose of the improvement.

Many municipalities have satisfactory ordinances and regulations, but fail to enforce them. Such ordinances usually require permits for connections to the sewers and provide for inspection of the connection by a representative of the municipality after the contractor has completed the construction work and before the trench is filled. Theoretically, this control should be sufficient, but too frequently the results are far from satisfactory. In many instances, the construction work is faulty, joints are made imperfectly, admitting ground-water to the sewer, the inspection is neglected or performed carelessly, improper wastes are admitted, and no proper record of the connection is maintained. When this system of control is started improperly, it is difficult to correct it and make it efficient, and usually the conditions become worse rather than better, until the sewer system is generally abused.

It seems necessary that municipalities provide a more immediate and direct control of the use of sewers, if sewerage systems are to be managed and maintained as they should be. The most logical and effective method of accomplishing this is the construction by the municipality of all connections to the public sewers from the building to the street sewer and the continuation of municipal control over such connections after they are constructed. The sewer department would organize its construction gangs for this work or would enter into annual contracts with responsible contractors, and the property owner would pay to the city the cost of construction, inspection, and recording.

When local treatment of industrial wastes is necessary to protect the sewerage system or sewage disposal works, such treatment would be provided by the industry and the effluent received into the sewer system under proper control. The connection could be equipped with an inspection hole to permit subsequent examination by the sewer department of wastes discharged through it, and the discharge of prohibited wastes could thus be detected. It is believed that this method would insure better construction of the connection at lower cost, would largely prevent the misuse of sewers, and would assist in securing efficient operation of sewage treatment processes. Incidentally, it would probably arouse a more lively interest on the part of the city officials in the management and maintenance of the sewerage systems and would likewise remind the public that the system is an important feature of the community development and must be controlled in a business-like manner if its value is to be realized.

DISCUSSION

HARRISON P. EDDY,* M. AM. SOC. C. E.—One item not mentioned by Mr. Hammond is important, namely, the effect of temperature on the period which should be provided for sludge storage in the digestion tank. Presumably, the eight months mentioned refers to the climatic conditions of Brooklyn, N. Y. In the north, digestion is confined in large measure to the warmer months of the summer, and storage of the sludge is required over a considerable part of the year. The same treatment in a warm climate where digestion can proceed throughout the year at a nearly uniform rate, may require a very different period of sludge storage, and, therefore, it would seem important to consider temperature conditions as well as the quantity of solids in the sewage.

The point presented by Mr. Stevenson in connection with the seasonal operation of sewage treatment plants is also important. Although, from some points of view, it is desirable to deal with this subject in an idealistic manner, it is probably not wise to disregard economic conditions. The following illustration may be cited to show that it is practical to operate a plant according to climatic and physical conditions. A certain plant on a relatively small stream was equipped with various means of treating industrial wastes. These wastes are first screened and then passed through sedimentation tanks, after which they are treated by chemical precipitation; and, finally, the treated liquor is passed through sand filters. On a branch of the stream there is a reservoir with a capacity of about 300 000 000 or 400 000 000 gal. For a number of years, this plant has been operated according to climatic conditions. In winter, screening and sedimentation have appeared to be a reasonable treatment; in the latter part of May, as the stream flow is reduced, the sand filters are put into use in order to treat a substantial part of the wastes, and by that improved treatment the river is maintained in a relatively good condition. Still, later, as the flow in the river is reduced further, water is drawn from the reservoir, and parts of the unfiltered wastes are diluted with this flow in proportion to their quantity. During the period of hot weather of August and much reduced flow of the river in September, it is customary to take advantage of chemical treatment, and the wastes are treated with sulphate of alumina. The treated liquor is allowed to settle, and a part of the effluent is passed through the sand filters, the remainder being diluted with water from the reservoir and discharged directly into the river. On one occasion, during an extremely dry season when the supply of water became exhausted, sodium nitrate was added to the effluent to supply oxygen to the stream. This is rather an elaborate treatment, but it is far less costly than treating all the wastes throughout the year, in the manner required for the unusual conditions of the summer season.

The subject of separate systems of sewerage and drainage, presented by Mr. Dittoe, is of surpassing importance. About thirty years ago, a Massachusetts city with a population of approximately 20 000 built a separate system of sewers with extreme care. The pipe joints were made very tight, and the

* Cons. Engr. (Metcalf and Eddy), Boston, Mass.

system was apparently an excellent piece of workmanship. In recent times, measurements of the dry-weather flow at the treatment plant have shown less than 300 000 gal. in 24 hours. In time of storm, the flow has risen nearly to 3 000 000 gal. in 24 hours. A thorough investigation to ascertain whether it was possible to correct that condition has shown that, except in the case of one or two sewers which could be and have been improved, it is practically impossible to better the situation.

In a certain part of another city of relatively large size, separate sewers and drains were built by a commission independent of the City Council. On their completion, they were turned over to the Board of Works with an explanation, that roof drains were not to be connected with the sewers, and that sewage was not to be discharged into the storm drains. Within a short time, sewage was discharged into the drains, and water from the roofs was discharged into the separate sewers. It has been practically impossible to prevent these conditions, notwithstanding the fact that the city has a City Engineer who undoubtedly knows the needs of the case, but is impotent to remedy the conditions.

A number of similar cases might be mentioned, where separate systems of sewers have been so overburdened with the discharge from roofs and even from streets, where the street-water connections have been made by the city, that they have become partial, if not complete, failures. The problem is so serious that it is questionable in many cases whether separate systems of sewers should be constructed. Undoubtedly, separate systems will be constructed in the future, and it is to be hoped that means will be devised for using the sewers and the drains only for the purposes for which they were built.

About thirty years ago, separate sewers were built in a certain part of Worcester, Mass., and the care and diligence required to assure their proper use is a serious task. It may be of interest to mention the routine by which it has been accomplished. The connections between the sewers in the buildings are made by drain layers who are licensed by the Board of Aldermen, on the written approval of the engineer in charge of the sewerage system, and occasions have arisen when that approval has been withheld. Connections are made under the direction of the engineer in charge who inspects the work. The plumbers are licensed by the local Board of Health and, before they make any plumbing changes or construct new plumbing, are required to file a drawing showing what they propose to do. That drawing is filed with the local Board of Health which turns it over to the Sewer Department the approval of which is required before a permit to do the work is issued. After the plumber has completed the work, it is inspected by the Department of Health. That, however, was not sufficient, because where both sewers and drains are required and the pipe lines are carried to the inside of the cellar wall by the drain layer, the plumber has no means of ascertaining which is the sewer and which is the drain. Finally, it was decided that the hub of the drain pipe should be painted white and that of the sewer pipe remain its natural black color. Even this complicated control did not completely elimi-

nate wrong connections, made either by mistake or surreptitiously; and thus far it would seem practically impossible to prevent all such connections. House owners, without leave or license, will connect roof waters with sanitary pipes, and it is almost impossible to ascertain where such connections have been made. Mr. Dittoe has certainly presented a very serious question for consideration.

For many years, it has been the practice at Worcester to construct the separate system in the outlying districts, and parts of the central part of the city has been re-sewered according to the separate plan. The speaker knows of no changes in the combined system since 1900, and he does not know of any further plans for changes to the separate system.

J. Frederick Jackson,* M. Am. Soc. C. E.—The speaker assumes that the object of this Symposium is to hasten the day when pollution will be removed from the streams in the United States. It has been stated that methods of sewage and sludge disposal should be adapted to the different classes of sewage being treated. As a general statement this is true, but engineers have not been successful in demonstrating this fact to the public at large, and it is still possible for any one with some new method to approach city officials having the final say in such matters and with a plausible story apparently convince them that they have the one panacea which will solve all sewage disposal problems.

The Industrial Wastes Board of Connecticut which was legislated out of office, was, strange to say, severely criticized, because it would not endorse a bill the provisions of which seemed so drastic as to make enforcement of them impossible.

One of the sensational papers in Connecticut contained a full-page article in which it was stated that a problem in stream pollution which the Industrial Wastes Board had had under investigation for three years and had failed to find a solution, had been solved in one week by a patented process controlled by a party outside the State. The reporter who prepared the article stated that while inspecting the plant, the patentee who was demonstrating the operation of the process, took a bottle of the wastes, added some ingredients, shook it up and down two or three times, and then holding it up to his view asked, "Are you satisfied?" He replied, "I am." Such ocular demonstrations of the solution of troubles of this kind apparently are effective with certain people.

The speaker believes that no solution of the problem of stream pollution will ever be accomplished in the United States, or elsewhere, until a strong public support is carefully and conscientiously built up before hand. There have been a number of remarkably careful and reliable investigations of stream pollution. What the public wants is a demonstration of some particular problem that has been solved or of some particular stream cleaned up. Investigations, no matter how thorough, apparently have little effect on legislatures. At the hearings before the Judiciary Committee in Connecticut, the Industrial Wastes Board was told, "investigations of stream pollution have

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been going on in this State for thirty years, your particular Committee has been investigating the problem for four years, and you have spent $75 000. What have you accomplished? Nothing. We think that there have been investigations enough. What we want is the removal of the pollution from the streams." The bill which was under discussion provided that all pollution of the streams of Connecticut should cease by 1923, and this bill would have been passed by the Legislature except that, as one of the Judiciary Committee stated, the members of the Committee could not frame it so that it would be operative. If members of the Judiciary Committee cannot frame a bill to meet a proposition of this kind, how is it expected that engineers are going to do so? Because the engineers of the Industrial Wastes Board refused to set any time limit as to when they thought the pollution of the streams of the State should cease, the members of the Committee said, "Then you have failed."

As a result of experience in Connecticut, the speaker would call attention to the inadvisability of delaying too long the issuance of reports. In most of the important investigations in this country on stream pollution, the report is cold by the time it is published, and it seems that if some method could be provided whereby reports or portions of them could be issued while the problem is alive and before the public, it would be of great assistance in effecting a solution.

F. A. Dallyn,* Esq.—One of the most encouraging signs in this discussion is the emphasis placed on the fact that sanitary engineers are now including biologists in their forces. The recognition of the part that biology is to play in the future of stream protection, in the operation of disposal plants, and in the degree of purification required, is the greatest step that has been taken by engineers in a long time, and it is hoped that a great deal of the time in future meetings of engineering societies will be left open to the researches of biologists. In his experience, the speaker has found no easier way of convincing industries of the facts relating to any situation than by putting them on a scientific basis. Nowadays, practically all large industries employ scientific experts; and when the matter is put before them in convincing terms which they understand, they appreciate the situation better than when engineers spoke in general terms of what they could do under such conditions.

W. F. Wells,† Esq.—As a biologist, the speaker will discuss briefly some phases of stream pollution, which are not necessarily engineering, but which are vitally connected with the problem of improving the streams in conserving the qualities of their waters. Mr. Jackson has brought up the point that although engineers have a great many data on the subject of the purification of wastes, when the public becomes interested in this subject as relating to the purity of water, it is at a loss to find any information which it can understand. In the last year or two, this question has become very important, because of the interest taken by those who are concerned in fish, commercially

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† Biologist and Sanitarian, State Conservation Comm., Albany, N. Y.

and for pleasure, as well as many other interests, such as bathing, gunning, etc., which are legitimate uses of these great resources of the people.

Recently, the speaker attended a conference in Washington, D. C., called by Herbert Hoover, M. Am. Soc. C. E., Secretary of Commerce, to consider problems brought about by the pollution of the coastal waters as affecting the fisheries. Incidentally, Senator Frelinghuysen, of New Jersey, pointed out the great importance of this subject, dwelling on it at considerable length as relating to the insurance business, namely, the higher rates on water-front property, which are brought about by oil pollution. Much has been heard about oil pollution recently. There are many phases and many angles connected with the problem of conserving the quality of the waters, and the people themselves are vitally interested, as shown whenever any one discusses the subject before those who are not particularly interested in the purification of wastes.

In fish conventions, the subject is given much attention, and, as Mr. Jackson has stated, the chances are that somebody will appear with a panacea for all the ills attendant on pollution; and the interest which these enlightened individuals take in the subject, and the ease with which they are convinced that there should be no such thing as pollution, is rather disheartening to one who is acquainted with the results which have attended such efforts in the past, and which are common knowledge among the Engineering Profession, but never get outside of it. The only way the question can be developed to the importance which it rightfully deserves, is by promoting in some manner public interest which is hungry for action of some sort, by uniting, as far as possible, the interests relating to the pollution of waters, by trying to get together those who are working on it and those who are interested in it, and by devising some means by which the knowledge that engineers have, may be given to the people who are concerned and whose resources are gradually being monopolized by individuals, industries, and particular communities. It is rather difficult to get such co-operation.

The speaker discussed the problem at the conference in Washington along that line, but his opinion did not receive much encouragement among officials, and he doubts whether it would receive the necessary attention among the Engineering Profession. However, the steps which are being taken, such as those mentioned by Professor Phelps, to develop a language of pollution—a technical language, terms in which the quality of waters can be measured and in which the purification of wastes can be stated—that will bring out the problem concretely, so that it can be handled, will tend, in the long run, to bring together those who are studying this subject, those working on studies of fish life, as well as those working along other lines. If they have some medium by which they can get into contact, ultimately the solution of stream pollution will be realized, and a great advance will be made not only with respect to general questions relating to the subject, but in the support that will be given the Engineering Profession in carrying out its essential work.

Alexander Potter,* Assoc. M. Am. Soc. C. E.—It is impossible to overemphasize the importance of securing tight joints in house laterals and of

* Cons. Engr., New York City.

eliminating all waste water not intended to be cared for by a sanitary sewer system.

Engineers too frequently have labored faithfully on the construction of the street mains of a sanitary sewer system only to find their efforts defeated afterward by the careless and indifferent construction of house connections. When these conditions occur, it is almost impossible later to correct them, short of reconstruction of most of the house laterals. The speaker recalls an effort at Summit, N. J., some years ago, to eliminate the leakage in the system, which came very largely from the house connections. Carelessness in house-connection construction is not immediately noticeable in a gravity system, but when, as in the case of Summit, it became necessary to change from a gravity system to a pumping system which operated against a 200-ft. head, the elimination of all leakage was essential.

Although the exclusion of house leaders made some reduction in the quantity of waste, the bulk of the leakage came directly from the open joints of the laterals themselves. That much of the infiltration in sanitary sewers comes in through manhole covers is a popular conception, but the two or three days' lag in abnormal flow after a storm indicates a more indirect source.

Efforts made by municipal engineers to provide tight sewers have been thwarted, often purposely by town officials in their endeavor to make improper use of sanitary sewers, that is, for cellar drainage, roof leaders, yard drainage, and here and there a street catch-basin. The protest of the engineer and his pointing out the disastrous conditions which inevitably result from the improper use of house sanitary sewers, have little or no effect in many cases on the officials responsible for the maintenance of the sewer.

In one municipality with which the speaker was identified, the officials deliberately attempted to secure storm sewers by connecting leaders, yard, and cellar drains to the sanitary sewer system. Expostulation only brought the retort: "The sooner the sanitary sewer is used to capacity, the sooner we will get an adequate system". This argument might be valid if the house laterals could be transferred from the sanitary sewers to the storm sewers at will.

This disregard of the purposes for which the sewer is built causes overflow at low points and along outlet lines long before the normal capacity of the sewer has been reached. The resultant pollution is tolerated for years and is often greater in volume than that contributed from the legitimate overflow from combined sewers during heavy rains.

The speaker, knowing that these conditions exist and that they are next to impossible to remedy, has believed for many years that the adoption of the combined sewers with dry-weather interceptors is preferable to the strictly sanitary sewer. In many cases, only sufficient funds are available for sanitary sewers. Then, of course, the adoption of the separate system is logical and proper.

As far as the effect of the final disposal of the sewage is concerned, the disadvantage of the greater fluctuation both in the quantity and character of the sewage reaching the disposal plant and of the occasional discharge of

diluted sewage at storm overflows is offset by the advantage of excluding from the watercourse the filth accumulating in the streets during protracted dry spells, which exclusion cannot be affected when sanitary sewers are used.

The speaker is pleased to note a change in the policy of the health departments of many States in giving their approval to the use of the combined system, and it is hoped that this policy will have a wider range.

EDWARD S. RANKIN,* M. AM. SOC. C. E.—As another instance of the difficulties encountered by the engineer in preventing the improper use of sewers, the speaker's experience in Newark, N. J., may be of interest.

All the older sewers of Newark, which comprise about two-thirds of the city system, are built on the combined plan, but, in a number of the outlying sections, the separate system has been used during the last 20 years.

On the separate system, the average citizen cannot understand why he should not be allowed the same privileges as the citizen on the combined system, and be permitted to drain roof water into the sewer. For some years one of the members of the old Board of Works was a plumber who apparently held the same views. An investigation, a few years ago, disclosed a large number of leaders connected with these sanitary sewers, several of them having been made by this City Official.

Under the present commission form of government, this matter is entirely controlled by the Engineering Department, and the practice referred to has been almost eliminated.

S. JOHN SCACCIAFERRO,† JUN. AM. SOC. C. E.—The speaker was much interested in the paper presented by Mr. Pearse. With reference to the oxygen requirements of fish life, the results of some recent experiments performed under the direction of Professor C. L. Walker, Affiliate, Am. Soc. C. E., at Cornell University, for the New York Milk Conference, may be of interest. The experiments in general indicate that the ordinary milk waste is not toxic to fish, and is not detrimental to them except as it utilizes available oxygen; as long as the oxygen content is not reduced below 1 part per million, the fish will continue to live. The fish used were bass, trout, and the ordinary fish found in the lakes of Central New York State. During October, the trout and bass were kept in undiluted milk waste, both raw and after septic tank treatment, without any evident detrimental effect. The oxygen content, however, was about 4 parts per million.

Regarding the bio-chemical oxygen-demand test, the speaker agrees with Mr. Pearse's statement as to the usefulness and value of the test; he thinks, however, that the modified "excess oxygen" method developed by Messrs. H. B. Hommon and E. J. Theriault, (the method used in Mr. Pearse's experiments), requires further study, modification, and improvement. This statement is prompted by the results of some experiments made in the Sanitary Laboratory of the College of Civil Engineering, Cornell University in 1920, the procedure described by Mr. Hommon‡ being carefully followed. The wastes used were:

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* Engr. in Chg., Bureau of Sewers, Newark, N. J.
† Civ. and San. Engr., Clifton, N. J.
‡ *Public Health Bulletin*, No. 97.

Milk-can and bottle washings before and after treatment in septic and Imhoff tanks, the effluents of these tanks after treatment in sand and percolating filters, cheddar cheese whey, and the sewage of Ithaca, N. Y., before and after septic tank treatment. Tests at 1, 2, 5, and 10-day incubations at 20° cent., were made of fifty-two series, each series consisting of three dilutions of a waste (except for the sand filter effluent, in which case the undiluted waste was used). Tap-water with an average initial oxygen content of 6.6 parts per million was used for dilution.

The results obtained were very discouraging, their outstanding features being a general inconsistency and the presence, at times, of greater amounts of dissolved oxygen on one day than on the day preceding. This last condition occurred frequently, despite the care taken to prevent any possible re-aeration or loss of iodine in the titrations. Due to the rather limited number of tests, and the wide variation in the results, no definite conclusions could be drawn. The results obtained, however, indicated that:

(1).—The "excess oxygen" method for the determination of the bio-chemical oxygen demand, although simple of performance, does not yield consistent results.

(2).—The bio-chemical oxygen demand is not independent of the dilution, as stated by Mr. Theriault,* but varies with the concentration of the waste, being higher for the lower concentrations.

(3).—No prediction of the probable demand at the longer incubation periods can be made from the results at the shorter periods (again failing to corroborate a statement of Mr. Theriault,* that from the results of the 5-day or possibly the 2-day incubation, the demand for the longer incubation periods could be predicted).

Despite the results obtained, the speaker still firmly believes that the test is important, and that further experiments are desirable, which will improve the "excess oxygen" method for the determination of the bio-chemical oxygen demand.

GLENN D. HOLMES,† M. AM. SOC. C. E.—The papers and discussions on Stream Pollution and Sewage Disposal have been of particular interest to the speaker as, at Syracuse, N. Y., these problems have been studied for some time for the purpose of improving the sanitary conditions of the watercourses throughout the city, and of Onondaga Lake into which these streams discharge.

The intercepting sewer system, on which construction was started some years ago, is practically completed. The watercourses which required deepening in order to permit interception of sewage from the lateral sewers without admitting back-water from the streams, have been improved and lined with concrete to insure permanency.

Tanks and fine screens of the types described by Mr. Hammond and Mr. Skinner, as well as many other processes, have been considered as methods of sewage treatment at Syracuse.

The sewage testing station, at which several types of settling tanks, filters, activated sludge units, grit-chambers, etc., were operated, furnished many data of pertinent value. These data together with those secured

* Public Health Report for May 7th, 1920.

† Chf. Engr., Intercepting Sewer Board, Syracuse, N. Y.

regarding sludge disposal, the volume, fluctuation of flow, and character of the sewage, as well as the character and condition of the water of the streams and lake, were of great assistance in reaching conclusions regarding the method of treatment best suited to the local conditions.

The method of treatment adopted includes degritting, coarse screening, and plain sedimentation in tanks equipped for continuous or frequent removal of sludge. The sludge is to be pumped about two miles to the lagoons of the Solvay Process Company and after intimate mixture with the waste liquors, will be precipitated and buried in the lime banks. These waste liquors are germicidal and are of sufficient volume to disinfect thoroughly more than ten times the quantity of sewage sludge which will be produced. The solids deposited from the industrial wastes are thirty to forty times greater in volume than the sludge solids, which insures complete and effective burial. This method of sludge disposal is unique and precludes the possibility of creating objectionable conditions in the vicinity of the lagoons, the treatment plant, or Onondaga Lake.

Before submitting the proposed method of treatment to the State Department of Health for approval (the approval has now been given), Harrison P. Eddy, M. Am. Soc. C. E., was asked to review the studies and conclusions and make such further investigations as he deemed necessary to enable him to report and recommend the method of treatment best suited to the sewage of Syracuse. It is gratifying to state that Mr. Eddy concurred in the recommendations.

In connection with the work at the testing station, an investigation was made to determine the relative effect of detention period and velocity of flow on the deposition of suspended solids. The detention period was found to be the controlling factor and velocities of flow within the limits of the experiments, which ranged between 12½ and 150 ft. per hour, had no effect on the clarification or removal of matter carried in suspension.

The percentage of removal is dependent to a large extent on the strength of the crude sewage, and this should be kept in mind when comparing or interpreting the results of treatment of different sewages. The term, "percentage of removal," is too frequently used without due consideration of the strength of the crude sewage.

A removal of 60% was obtained in ¾ hour with Syracuse sewage which contains an average of about 270 parts of suspended solids per million. The contributing factor in the attainment of this high efficiency, with the comparatively short detention period, was the frequent removal of the sludge, precluding septic action.

Rudolph Hering,* M. Am. Soc. C. E.—With reference to the discussion by Mr. Eddy, the speaker was asked, about 41 years ago, to report on the European sewerage systems for the National Board of Health, largely with reference to obtaining information regarding the separate system. Only two such systems were found in Europe, and these were for the Cities of Oxford and Reading in England. Continental engineers did not believe in the exclusion of rain water from sewers, much on the grounds stated by Mr. Eddy. In England,

* Cons. Engr., New York City.

some discouragement was felt even by the officials of the two cities mentioned, one of whom stated that there was trouble in preventing the discharge of rain water into sewer pipes, and also the surreptitious discharge of sewage into the storm drains. The City Engineer of Oxford, Mr. White, stated that if he had it to do over again, he would use the combined system which was common in most other European cities.

In his report, the speaker distinctly discriminated between these two systems with special reference to their application in the United States. It was found proper to place these systems on a par. The application of the separate system has large possibilities in the United States, as compared with Europe, and engineers should not recommend the combined system indiscriminately for all cities. Many communities could not afford the expense of the combined system and were satisfied to start with a system for sewage removal alone, with great benefit to their comfort and health. In New Orleans, La., a complete double system was recommended at the outset as it was found more economical than the combined system, because it cost less to pump the small volume of sewage into the Mississippi River, even at high water, and to let the large quantity of water from the heavy rainfalls flow in the other direction into Lake Pontchartrain at ocean level.

The speaker would recommend the separate system for many of the cities of the United States, particularly the smaller ones, for sanitary reasons, for otherwise they might not have any sewers until they could pay for the combined system.

About 35 years ago, the speaker was asked to study the Chicago sewage problem which had developed into a very serious situation. The question was whether to discharge the sewage into Lake Michigan, to purify it on land, or to discharge it into the Illinois River. At that time, little was known about bacteria. In fact, the first knowledge about bacteria in relation to sewage was acquired about the time the Chicago studies were being made. The available information, however, was not sufficient to aid in the solution of the problem.

In the United States, only the chemical analysis of sewage was generally reported, which by itself is not satisfactory. The speaker felt that the dilution question of disposal was an important one to consider for Chicago, not only on account of a possible lake disposal, but chiefly for the proposed channel which was to carry the diluted sewage to the Mississippi River. Information was available from a few places in Europe, particularly Paris, Hamburg, Dresden, Magdeburg, and a large number of cities in England. The best information, however, was from the work done on the Blackstone River in Massachusetts, and on the Desplaines River, into which the Chicago sewage was to be discharged, which information determined how much sewage could be put in running water without becoming objectionable.

All this information from Europe, Massachusetts, and Illinois was then plotted. It was astonishing to note the correspondence of these different data, and the indication from all of them that a dilution of from 2½ to 5 cu. ft. of water per sec. per 1 000 people appeared to be satisfactory.

As a result of this investigation, it was recommended for Chicago that there should be introduced into the Drainage Canal about 4 cu. ft. of water per sec. per 1 000 people sewering into the Canal. The speaker felt that he was not making a mistake in recommending an expenditure of $30 000 000 to have this channel built large enough to effect the dilution mentioned. In view of the slower velocity of water in the Chicago River and, consequently, the settlement of suspended sewage matter, a frequent removal by dredging was stipulated.

C. M. Baker,* Esq.—The abuse of sewers is a problem which cannot be over-emphasized. In practically all States it is the custom for the State boards of health to require the separate systems. In Wisconsin, there has been in operation for 7 or 8 years a system of plumbing inspection. This system was started before the Bureau of Sanitary Engineering was established.

The Department has a certain number of State inspectors, and all the plumbers in the State are required to secure State licenses. Through this organization a complete system has been built up, and the situation is fairly well under control.

Mr. Jackson is right when he states that the people want results in stream pollution rather than continued investigation as to the extent of the pollution.

The Chairman of the Conservation Commission of the State of Wisconsin has stated that at a National meeting of the representatives of the different conservation commissions of the States, he recommended that legislation be enacted prohibiting the discharge of polluting wastes into the streams —one of those suggestions for radical legislation.

The problem of industrial wastes has only been touched on, and very definite steps should be taken toward abating the existing polluted conditions. It seems to the speaker that possibly some of the funds expended in investigating existing conditions, the extent of the pollution, etc., might well be used for devising ways and means of eliminating or partly eliminating, at least, some of the most critical conditions with reference to pollution.

P. H. Norcross,† M. Am. Soc. C. E.—The speaker concurs in the statements, made by Mr. Hatton, regarding the necessity of disposing properly of the residual sludge from disposal plants. Experience has shown that the usual method of sludge disposal is primarily the cause of adverse criticism from laymen and the public at large.

Atlanta, Ga., was the first large city in the United States to construct a sewage treatment plant using Imhoff tanks for digesting sludge. The city now has three plants with capacities of 3 000 000, 5 000 000, and 6 000 000 gal. daily, respectively. The largest plant is subject to some of the abuses and troubles caused by the misuse of sewers, and, in addition, its capacity is overtaxed. The dumping of the sludge in low areas surrounding this plant has tended to create a nuisance and to invite litigation against the city.

Leonard S. Doten,‡ M. Am. Soc. C. E. (by letter).—Since sewage is one of the chief causes of stream pollution, it has become customary to associate the

* State San. Engr., State Board of Health, Madison, Wis.
† Cons. Engr., Atlanta, Ga.
‡ Capt., Q. M. C., U. S. A., Chicago, Ill.

terms "stream pollution" and "sewage treatment", although they are antithical in meaning. All organic wastes, whether or not of human origin, industrial wastes, and the waste waters of mines, are the important contributing factors to the pollution of streams, lakes, and other natural bodies of water.

The introduction of water-carriage sewerage systems has greatly intensified the pollution of such bodies of water. For many years, it was the universal practice to discharge both storm water and domestic sewage into natural watercourses, rivers, lakes, or harbors. As a result, these bodies of water, in the vicinity of large centers of population, became greatly polluted, and, in some instances, very foul. Small streams have become so polluted as to be nothing less than open sewers. Such practice is no longer justified, as effective methods for the treatment of sewage and industrial wastes are now available. Notwithstanding the progress that has been made in the art of treating such wastes, writers of treatises on sewage disposal devote much space to discussions on "disposal by dilution." This term is synonymous with "stream pollution." So long as sanitary engineers sanction the practice of loading streams with organic wastes "to the limit of toleration," little progress will be made in the elimination of stream pollution. There is great need for reform in this matter.

Engineers and sanitarians should start a movement for clean streams. Undoubtedly, it will be a difficult task to educate the public to appreciate the higher standard and its advantages. The trend now seems favorable, although not as positive as it should be. The educational work should be along three important lines: public health, industrial, and economic.

It is essential that streams and lakes be protected against pollution, as in the future, all large public water supplies are, or must be, derived from surface sources. There will be an increasing demand for larger supplies, due to higher per capita rates of consumption, especially as the cities and towns become more populous. As a consequence, some of the streams which are now becoming more and more polluted by sewage and industrial wastes will ultimately be required for water supply and industrial purposes. With this condition confronting us and the trend of engineering opinion becoming more averse to the use of "reconstructed" water for domestic supplies, it is not difficult to see that the time is coming when the treatment of sewage and industrial wastes will become practically general in the United States.

Some of the States have adopted effective laws for the protection of streams against pollution. The State boards of health and other organizations having administrative authority to carry out these laws, should be aided by the Engineering Profession in every possible way.

Although there are several effective methods now in use for the treatment of sewage, the results are not in all cases satisfactory. The need is for standardization of practice, both in the design and the methods of operation of plants. Advancement in this work, like that in many other fields, must be made largely by perfecting details of existing processes or methods. Owing to the nature of the problem, it is extremely improbable that any process will be discovered, which will revolutionize the art.

Modern sewage disposal plants are not only "sewage treatment" plants, but can be operated so as to be in reality "sewage purification" plants, using the term, purification, in a relative sense, as it cannot be used in the absolute, even in water purification.

As generally understood to-day, sewage treatment accomplishes three objects: The removal of suspended matter, the reduction of that matter to a stable condition, and the oxidation of organic matter carried in solution. Treatment plants, therefore, should be designed so as to secure these results in the most satisfactory manner and at the least cost.

The sewage tanks now in use are highly efficient in the removal of suspended matter. It is not only possible to remove practically all settleable matter, but also a large percentage of the colloids.

The oxidation of organic matter in solution in the clarified liquid can be accomplished either by filters or by further tank treatment using the "activated sludge" principle.

As the non-settleable suspended organic matter (colloids) in sewage is a factor of considerable importance, even amounting to as much as 30% of the total matter in suspension, it is important that a large quantity of this matter be removed by tank treatment before the secondary, or oxidation, process is begun. Plain sedimentation, obviously, is not effective in the precipitation of colloidal matter. It is necessary, therefore, in treating sewage containing a large percentage of matter in this form, to use a type of tank in which the processes of "bacterial coagulation" and sedimentation may be carried out most completely. These are Nature's processes of clarification. A good illustration of the advantages of this method of treatment over plain sedimentation may be found in the clarification of laundry wastes. Plain sedimentation alone has no appreciable effect in lessening the turbidity of the liquid. Such wastes may be left in a quiescent condition for days without showing evidences of clarification. On the other hand, laundry wastes may be very effectually clarified by passage through a tank in which the biological processes are well established and conditions are favorable to the precipitation of the flocculated matter. These wastes may constitute 75%, or even more, of the total sewage without deleterious effects on the effluent, if a suitable range of detention periods is maintained.

John A. Griffin,* M. Am. Soc. C. E. (by letter).—The subject of stream pollution and sewage disposal is of particular interest to the writer at this time, in that the City of Los Angeles is confronted with the necessity of resorting to an emergency measure to cope with its overtaxed outfall sewer.

Los Angeles is one of the largest cities using a separate system of sewers, and the sewage is a relatively "strong" sewage. The diversion or disposal thereof into a relatively small creek which contains, in the dry season, a limited quantity of water, might tend to intensify the pollution of the creek. Practically, no water supply is derived from this creek and only a few wells are in proximity thereto. The level of the water in the creek is slightly lower than the ground-water in the surrounding areas, which would tend to drain

* City Engr., Los Angeles, Cal.

waters from the land and wells to the creek rather than pollute the wells from the stream. However, pollution to any marked degree would become objectionable, due to the fact that the course of the creek is through a rapidly growing suburb.

The isolation of the creek at other points than Culver City, makes its use, as an emergency measure, safe and justifiable, although it is well recognized that a dilution of from 2½ to 5 cu. ft. of water per sec. per 1 000 people sewered is necessary to prove satisfactory under ordinary conditions, yet the writer believes that a much smaller dilution factor can be resorted to with safety in this instance. If experience should demonstrate otherwise, it will only be necessary to discharge in the head-waters of this creek, sufficient fresh water from the Los Angeles domestic supply to bring the dilution factor to a point within the limit of safety.

The rapid growth of the city has brought about a condition that is causing much trouble. The outfall sewer to the sea, constructed in 1907, has become overtaxed to the extent that, in the dry-weather period, sewage has begun to flow out of the manholes into the streets and in the wet-weather period the discharge into the streets is enormous. This condition was anticipated nearly two years ago, and a bond issue was submitted to the people. This bond issue was defeated on June 7th, 1921.

With the overflow from the manholes into the public streets in a thickly populated section, came the apparent menace of epidemic and the necessity for immediate action. Although this overflow ultimately found its way into Ballona Creek over improved streets, it was necessary to provide a means to divert the surplus sewage to the creek without allowing it to flow along the streets. Permission to make this diversion has not been granted by the State Board of Health for the reasons previously mentioned, that is, the pollution of the small quantity of creek water, nor has sanction or permission been given for the use of the streets as natural channels to lead the sewage to the creek. In the absence of such authority, it was necessary, from a public health standpoint, to choose the lesser of two evils, and tap the outfall sewer near the head-waters of Ballona Creek to permit the surplus to flow directly thereinto without rising to the streets. This was done on January 4th, 1922, with the result that injunction suits have been filed by people down the creek.

The writer thinks that members of the Society may be interested in the peculiar position of the State Board of Health, that is, to grant a permit to use the creek for sewage disposal, even as an emergency measure, means the creating of a nuisance, with possible damage suits, whereas the ordering of the city not to utilize the creek would result in using the public streets as sewage channels, with the consequent menace to the health of a considerable population.

The time required to put through any semi-permanent relief will be more than two years. The outlook is not good for quick relief, and although the writer has recommended that no extensions to the sewer system be made and that house connections to existing sewers be disconnected and cesspools be resorted to until a permanent relief is provided, it is doubtful whether this program can be effectively executed, due to legal and other difficulties.

Bacteriological tests of the water of Ballona Creek, at Culver City, indicate no considerable pollution, even with raw sewage being discharged in the headwaters. When the sewage is fine screened and a small quantity of chlorine added, it is anticipated that no menace to health will exist. This meager emergency treatment will be carried on until permanent relief has been provided for the outfall. The time required for the sewage to flow from the proposed plant to Culver City is six hours and, under normal weather conditions, oxidation will be a material factor in its purification. Beyond Culver City, practically no nuisance will exist with ultimate disposal in the tide-waters at Del Rey Beach.

CLARENCE E. KEEFER,* JUN., AM. SOC. C. E. (by letter).—The paper by Mr. Dittoe should appeal to those in charge of the maintenance of sewerage systems. Baltimore, Md., is a striking example of a city with a separate system of sewers, where efforts have been made successfully by the city authorities to have these sewers put to their proper use. This condition is accomplished by co-operation between the Sewer Division and the Health Department of the city, and the Master Plumbers' Association.

The municipal laws require that, before the plumbing system of a house is connected to the city sewers, a permit must be obtained from the Health Department. The connection to the sewer is made by a master plumber and inspected by an authorized city employee. If it is found that there are any rain leaders emptying into the sanitary sewers, they are immediately ordered to be disconnected. In those parts of the city that are unsewered, the sewage is usually collected in cesspools. When sewers are built in such sections of the city, it is usually necessary to re-arrange the plumbing in the houses; and before this work is finally completed, inspections are made to see that it is done properly. Occasionally, it is brought to the attention of the Sewer Division that either cellar drains or rain leaders are connected to the sanitary sewers. Under such conditions, an investigation is made, and if the work was done by a master plumber, he is warned not to do so again, otherwise his license would be revoked.

In addition to laws governing the control of rain water entering sanitary sewers, there are other laws which make it illegal to dispose of gasoline, kerosene, germicides, or solid wastes into these sewers. As Mr. Dittoe has stated, more is required than the passing of laws governing the misuse of sewers. It is essential that these laws are enforced, and until this is done the best results will not usually be obtained.

One thing which should be provided for by cities is a law requiring property owners to connect their plumbing to the sewers. When a sewerage system is built, a few property owners, for one reason or another, will either defer making their connections to the sewers or fail to do so. In Baltimore, the property owners are notified when the sewers adjacent to their houses are completed, and they are given thirty days, with a reasonable extension of time, to connect their plumbing to these sewers. Usually, there is no trouble in getting them to do this. In case they should make any prolonged delay,

* Asst. Designing Engr., Sewer Div., Baltimore, Md.

however, the work is done by the city authorities, and the property owner is billed or assessed for the improvement made.

In his paper Mr. Hammond speaks of the use of separate sludge digestion tanks in the treatment of sewage. As is generally known, these tanks have been in use at Baltimore for about 10 or 12 years, and a great deal of information has been collected to date. However, much additional work will have to be done before the design of sludge digestion tanks is perfected. The first tanks built at Baltimore were rectangular in shape, each being 140 by 103 ft. in plan and from 13 ft. 6 in. to 15 ft. 6 in. deep. The sludge enters these tanks through a 12-in. cast-iron pipe at the top of the tanks. Three inlets are provided for each tank. On the opposite side from the inlet pipe, there are three outlets at the bottom of the tank. When these tanks were designed, it was thought that, as the sludge digested, it would settle to the bottom, while the undigested sludge would remain at the top. Until August, 1920, the reverse condition prevailed. Practically all the digested sludge collected at the top, in the form of a heavy scum which was fairly dry. In fact, at times it was sufficiently hard to bear the weight of a person in walking. On top of this scum, grass, weeds, and other vegetation grew to a height of a foot or more. The sludge as it entered the tank sank through this scum, and flowed under it directly to the outlets. The horizontal distance of flow was about 100 to 125 ft. No figures are available showing the time it took for the sludge to arrive at the outlet, but, in many cases, it was only partly digested. The moisture content of the scum was approximately 70%, whereas that of the sludge beneath was approximately 80 per cent. In order to move the sludge from these tanks through the 1 100 to 1 900 ft. of 12-in. cast-iron pipe line to the sludge drying beds, it was necessary to break up the scum with a stream of water.

In addition to these rectangular tanks, there are sixteen cylindrical tanks, each 38 ft. in diameter and 24 ft. 6 in. deep. Each tank has a capacity of 20 610 cu. ft., with inlet and outlet pipes, 8 in. in diameter. The inlet pipes are at the tops of the tanks, and the outlets are at the bottom. As in the rectangular tanks, the digested sludge forms a scum on the top, and the undigested sludge remains at the bottom. An additional sludge digestion tank has been built, in which various improvements over the old tanks have been embodied.

As the problem stands to date, it is possible to formulate certain principles pertaining to the design of sludge digestion tanks at the Baltimore plant. The depth of the tanks does not govern the degree and speed of sludge digestion. At present, designs are under way for three additional tanks which will have an approximate capacity of 650 000 cu. ft. In the future, it is likely that the tanks at Baltimore will have a depth varying from 15 to 20 ft. It is natural that the topography of the location of a sludge tank should be one feature governing the depth of these tanks. Where there is sufficient land available, and conditions warrant it, there is no reason why long shallow tanks, such as those at Birmingham, England, cannot be used.

One fact that has impressed itself on those connected with the operation of the sludge tanks at Baltimore is that the inlets should be as far away as

possible from the outlets, in order to prevent the short circuiting of the sludge, which condition has frequently occurred. One obvious and simple method of doing this is to construct in the tank a series of vertical baffles around which the sludge can flow to the outlet. The inlets should be designed so that the incoming sludge can enter below the surface of the sludge in the tanks. This is necessary, because if the incoming undigested sludge is allowed to spill through the air, disagreeable odors will be disseminated. In the last sludge tank built, the outlets were placed at various elevations so that there would be a possibility of drawing off the digested sludge at different elevations.

The capacities of sludge tanks should depend largely on the quantity of material to be handled. The cylindrical tanks, of which there are sixteen in Baltimore, are thought to be too small for the large plant there. The maximum size of sludge tanks will depend on the quantity of sludge to be handled. At Baltimore, it is not likely that tanks will be built having a capacity of more than 250 000 cu. ft. One necessary appurtenance to a sludge tank is a flushing water line so that water can be added, in case scum forms, and the scum broken.

In 1921, an unexpected condition occurred in the sludge digestion tanks at Baltimore. In one of the rectangular tanks, gas action which was prevalent over the entire surface of the tank took place, and, in a short time, all the scum had sunk. Sludge from this tank was immediately put into the other two digestion tanks, and, before cold weather began, the scum had settled to the bottom, and gassing was occurring in all three tanks. When this gassing started, there seemed to be an intimate mixture of the digested and the undigested sludge, with a consequent increase in digestion. One other condition noted was that this material flowed readily through the 12-in. cast-iron, pipe line to the drying beds, frequently without having to be pumped. Furthermore, it was no longer necessary to add water in order to break up the scum so that it would flow to the pump. Whether this gassing action will start again is problematical, but every effort will be made to study its causes and have it occur if possible. Much remains to be done on the investigation of sludge digestion, in fact, various experiments have been conducted already on sludge digestion at Baltimore.

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WATER SUPPLY AND WATER PURIFICATION*

A SYMPOSIUM

By Messrs. George C. Whipple, Allen Hazen, C. A. Emerson, Jr., C.-E. A. Winslow, C. A. Holmquist, Robert Spurr Weston and Samuel A. Greeley.

With Discussion by Messrs. Louis L. Tribus, G. F. Catlett, John R. Baylis, P. H. Norcross, H. Malcolm Pirnie, C. M. Baker, George W. Simons, Jr., M. N. Baker, H. N. Bundesen, and Charles Haydock.

* Presented at the meeting of November 17th, 1921.

HISTORY OF WATER PURIFICATION

By George C. Whipple,* M. Am. Soc. C. E.

The history of water purification in the United States is intimately connected with the general movement to improve sanitation and public health, as well as with the rise of modern science. No excuse, therefore, is necessary for referring to these subjects by way of introduction.

The great sanitary awakening occurred in England about the middle of the Nineteenth Century. Following the social uplift, sometimes called the New Humanity, which occurred in the Twenties and Thirties, came the sanitary surveys of Sir Robert Rawlinson, the public health activities of Sir John Simon, the statistical studies of Edwin Chadwick, and, in the Forties, the medical investigations of Dr. Southward Smith. The English public health system dates from 1848. The new public health inspiration crossed the Atlantic and, in the late Forties, occurred the investigations of Lemuel Shattuck, which culminated in that remarkable document, the Report of the Massachusetts Sanitary Commission of 1850.† The movement, however, became side-tracked in the United States, because more important issues were before the public.

During and after the Civil War, the problems of sanitation were attacked anew. A Citizens' Association made an investigation of the sanitary condition of New York City in 1865. The Massachusetts State Board of Health was established in 1869. The American Association of Public Health was founded in 1872. Meantime, a new demand for public health measures had arisen. The Civil War had left an aftermath of disease. In particular, typhoid fever which had been so prevalent in the armies, was scattered throughout the country. Many a returned soldier had become unwittingly a typhoid carrier, and the typhoid fever death rates were very high. In Boston, Mass., from 1855 to 1860, the typhoid rates were between 40 and 50 per 100 000; during the Sixties, they were above 60 for one-third of the time; in 1872, the rate was 86 per 100 000—high figures for a city like Boston. The rates in other cities were equally high, sometimes much higher.

It should be remembered, also, that it was during the last half of the Nineteenth Century that modern science began its remarkable development. Several events marked the new era, the most important perhaps being the publication of Darwin's "Origin of Species" in 1859. With it, came the wide application of the inductive system of reasoning, of the experimental method. All sciences went forward as a result of the impetus given by the new ideas. In the late Sixties and early Seventies, Pasteur made his great discoveries. These were followed by the work of Koch and others in the Eighties, and the science of bacteriology soon arose. The experimental method was at once applied to the field of sanitation, including water purification. Although filtration had been used for a part of the London supply as early as 1829, the benefits of the process were not appreciated, nor its action fully understood for many years after that date. Following a cholera epidemic in 1839,

* Prof., San. Eng., Harvard Univ., Cambridge, Mass.

† Long unavailable, but now reprinted in "State Sanitation", Harvard University Press.

filtration was extended rapidly, and, in 1855, was made compulsory for the London Metropolitan District. It was the rise of the science of bacteriology and all that went with it, that caused the modern development of this greatest of the sanitary arts—the art of water purification.

Fifty years ago, the quality of the existing water supplies in the United States was low, judged by modern standards. Clearness and freedom from color, taste, and odor were the ruling standards, and even these were often not complied with. In some regions, a great deal of attention was being given to lead poisoning and to hardness. Water analysis was confined to the mineral constituents. The germ theory of the transmission of disease through the agency of sewage polluted water had not arisen. In the United States, water purification was practically an unknown art.

Beginnings of Filtration in the United States.—In 1866, the late James B. Kirkwood, Past-President, Am. Soc. C. E., was sent to Europe by the city authorities of St. Louis, Mo., to study the new methods which had been coming into use in England and Germany, for besides the London filters there were, at this time, filters in many of the English, and in most of the large German, cities. Kirkwood's report, the first important document on filtration published in the United States, may be said to mark the beginning of the history of water purification in America. His plan for filtering the muddy water of the Mississippi River, which was based on his European observations, was not adopted by the municipal authorities of St. Louis. It was fortunate that the plant was not built, for later studies of the filtration of such waters have shown that it certainly would have failed. Several other filters, however, were built as a result of his report, notably the filter at Poughkeepsie, N. Y., which was constructed in 1873 to clarify the water of the Hudson River. This filter was the most successful of several which were built about that time.

In 1878, Professor William Ripley Nichols went to Europe to study water purification for the Massachusetts State Board of Health and his report was published in the Annual Report of the Board of that year. Five years later, his notable book on "Water Supply" was issued, in which he discussed not only the problem of filtration, but that of tastes and odors as related to the algæ. In many ways, this book was a pioneer in its field. For a considerable time the center of interest in matters connected with water purification was in Massachusetts. In 1887, the State Board of Health established an experiment station at Lawrence, and the purification of water as well as the treatment of sewage was studied from all possible angles, engineers, chemists, and biologists contributing to the investigations. The experiments were made on the relatively clear but polluted water of the Merrimac River. Some of the men prominent in connection with this work were the late Hiram F. Mills, Hon. M. Am. Soc. C. E., the late Frederic P. Stearns, Past-President, Am. Soc. C. E., Allen Hazen and George W. Fuller, Members, Am. Soc. C. E., H. W. Clark, Edwin O. Jordan, Mrs. Ellen H. Richards, Professor Thomas M. Drown, and Professor William T. Sedgwick, the last two acting in an advisory capacity. This was the first important scientific study of the subject of water purification in America, and its results did much to develop the art.

While these experiments were under way, a notable epidemic of typhoid fever swept down the Merrimac Valley and included the City of Lawrence which took its water supply from the river. It soon became evident that a filter was needed, and, in 1893, a sand filter designed by Mr. Mills was put into operation. This filter was built as a single bed without a roof and involved certain complicated arrangements of coarse and fine sand, ideas which have not been followed in other plants; in fact, the filter has been largely rebuilt in recent years. This filter became a great object lesson to the country, as it showed conclusively that polluted water could be made safe for drinking. It was a practical demonstration of the ideas which had been developed at the Lawrence Experiment Station.

After his work at Lawrence and after conducting experiments at the World's Fair in Chicago, Ill., Mr. Hazen spent a year in Europe studying foreign practice, and from this experience came his book on "Filtration", which for a generation was the leading authority on the subject. Unfortunately, it is now out of print.

Meanwhile, filtration had been developing in another direction, in regions of the country where the water supplies were muddy. In 1883, the late Charles Hermany, Past-President, Am. Soc. C. E., conducted some experiments in the filtration of the water supply at Louisville, Ky. In 1884, Alpheus Hyatt obtained a patent on the use of sulphate of alumina as a coagulant, and in the same year a mechanical filter in which such a coagulant was used, was built at Somerville, N. J. Although mechanical filters had previously been used for clarifying water for industrial purposes, this was the first application of the method to a public water supply. The name of Professor Albert R. Leeds should be remembered in connection with the early development of this process, as he was its inventor. A mechanical filter was built for the water supply of New Orleans, La., but failed to meet the requirements and was removed at great loss to the promoting company. Several other installations were made in different parts of the country. Some of these early mechanical filters gave fairly good results, but others failed. It became evident that the process was not only useful but necessary for the purification of muddy waters, and also that engineers had not learned how to utilize it successfully. Then, Science came to the rescue. In 1893, the late Edmund B. Weston, M. Am. Soc. C. E., made some tests of mechanical filters for the City of Providence, R. I., which should be remembered as the first carefully conducted tests with this type of filtration. From 1895 to 1897, Mr. Hermany again studied the problem of filtration, this time with the assistance of Mr. George W. Fuller and a corps of engineers, chemists, and bacteriologists, acting for the Louisville Water Company and several companies interested in the construction of mechanical filters. These experiments, conducted on the water of the Ohio River, resulted in a notable report published by Mr. Fuller. The Louisville experiments were devoted to a study of the theory of the filtration of clay-bearing waters and the use of coagulation and sedimentation in connection with mechanical filtration. The principles thus established have served as the basis of the mechanical filters which have

since been constructed in the United States. Various devices used in mechanical filtration were also tested at Louisville and their weaknesses pointed out.

The experiments at Lawrence and Louisville demonstrated the fact that different kinds of waters require different methods of purification; that although slow sand filtration can be used successfully with a relatively clear water like that of the Merrimac River, at Lawrence, coagulation must accompany filtration in the case of muddy waters, such as those of the Ohio and Mississippi Rivers. With this thought in mind there followed a series of tests in different parts of the country. At Pittsburgh, Pa., Mr. Hazen conducted experiments to compare the relative advantages of sand and mechanical filtration for the water of the Allegheny River, and his report thereon was published in 1899. Mr. Fuller made further experiments in mechanical filtration in Cincinnati, Ohio, in 1898, and experiments were also made at Washington, D. C., Superior, Wis., New Orleans, La., Philadelphia, Pa., Reading, Pa., Boston, and elsewhere. It was a time of experimentation, and much of this experimental work was done by men who had received their early training in Massachusetts.

Progress of Filtration.—The first large slow sand filter to be constructed after that built at Lawrence, was designed by Mr. Hazen for the City of Albany, N. Y., to treat the water of the Hudson River. This filter was put into service in 1899. It differed in several respects from that at Lawrence, especially by having a masonry cover, experience having already shown that cold weather interfered with the operation of open filters like those at Poughkeepsie and Lawrence. The Albany filter has served in many ways as a model for the sand filters which have since been constructed in the United States, such as those at Washington, D. C., Philadelphia, Pa., Providence, R. I., and many other cities.

The first important mechanical filter to be constructed along modern lines was that of the East Jersey Water Company, at Little Falls, N. J., which was put into service in 1902 to purify the water of the Passaic River. As long as the basic Hyatt patent was in force, the construction of mechanical filters had been confined chiefly to those built by private companies; after the expiration of this patent in 1901, however, the art of mechanical filtration advanced rapidly. In 1904, a large mechanical filter was built for the Hackensack Water Company in New Jersey, and many others followed.

It will be seen, therefore, that it was just about the beginning of the Twentieth Century when water purification in the United States really began to advance as a practical measure. The last decade of the Nineteenth Century had been one of experimentation. The work done by the chemists and bacteriologists was to be taken up by the engineers and pushed rapidly.

Since 1900, there has been a great increase in the use of filtration. In 1870, practically no filtered water was in use in this country. In 1880, 30 000 people in cities having populations of more than 2 500 were using filtered water. In 1890, this number was increased to 310 000; in 1900, to 1 860 000; in 1910, to 10 805 000; and, in 1920, to at least 20 000 000. At the present time, more than one-third of the people living in cities which contain

2 500 or more inhabitants are using water which has been filtered. The number of filters in the country is probably not far from 800.

Present Problems.—Water purification would not be a growing art unless there was new work ahead, and unless new problems were being studied. A survey of the situation reveals some interesting facts and tendencies.

In the first place, not all the water supplies of the country are being adequately protected. There are still some places where grossly polluted water is being distributed to the people, although year by year these places are decreasing in number. There are many places where the water is reasonably safe, but is not attractive at all times for one reason or another. Fifty years ago, and even twenty-five years ago, the New England water supplies were on the whole better in quality than those in other parts of the country, but, to-day, filtration has become so common in regions where the streams are naturally muddy that the New England supplies, few of which are filtered, suffer in comparison with filtered waters elsewhere, in so far as the quality of attractiveness is concerned. In New England, the policy of utilizing natural storage and preventing pollution in all possible ways was wisely adopted, but these measures have their limits and, with the increasing populations, the filtration of surface waters is likely to become universal; it may even be required by law.

There are many places in the United States where disinfection has been adopted without filtration or in place of adequate attempts to prevent pollution. Gradually, the weakness of this policy is being discovered. Disinfection is very cheap, but cheap disinfection does not accomplish what filtration does, and it should not be accepted as a substitute for vigilance in protecting a water supply against pollution.

Since the typhoid fever rates of the country have been greatly reduced, consumers are thinking less about the dangers of their water supplies and more about its physical appearance, about its corrosive properties, and about its fitness for industrial uses, that is, they are seeking refinements. Thus, a new class of problems, such as water softening with the use of lime and soda, or with permutit, the removal of iron and manganese, and the neutralization of acidity, is receiving the attention of water chemists.

Many of the filters built 10 and 20 years ago have been in use long enough to show defects in practical operation, and experience has indicated wherein changes can be made to avoid depreciation, reduce repair costs, save money in operation, and get better results. Filter designs are slowly being modified as a result of these findings. It is natural that, at the time of building these filters, officials should give the greatest attention to construction cost, but, as time goes on, it is the costs of operation which are scrutinized. The usual lower first cost of mechanical filters is a reason why it is sometimes easier to induce cities to construct that type of filter rather than a slow sand filter which usually costs more to build but less to operate. Higher operating costs, however, make mechanical filters less popular after a term of years. The economics of filter operation can now be studied more carefully than was possible twenty-five years ago, and this is a phase of the subject which is likely to receive increasing attention.

Methods of testing the performance of filters by the use of analyses have not changed greatly in twenty-five years. In the early days, stress was laid on the percentage removal of bacteria, color, turbidity, and organic matter. Tests were made to show what the filter was able to do. Engineers now are more interested in knowing what the filter actually does, and the analyses must serve as a test of the filter operator as well as the filter itself. When discussing water analyses, Dr. Drown used to say that "a state of change is a state of danger." Now, the principle is emphasized that a state of irregularity is a state of danger. Filters are wanted, which operate so as to give a uniform product from day to day and hour to hour. Therefore, attention is being given to the application of the mathematical theories of probability to filter records. There are great possibilities in this study, but they are appreciated at present by only a few engineers.

A new impetus is being given to many of the problems of water purification by the use of new and simple methods of determining the true acidity of water in terms of the hydrogen ion concentration. This and other methods of physical chemistry are likely to clarify many ideas in regard to problems of coagulation, corrosion, disinfection, and the growth of algæ.

Bacteriological tests of water have never been as definite as chemical tests. The difficulties have been partly analytical and partly statistical. The test for *B. coli* has been largely, and, on the whole, successfully, used; but it has many shortcomings. The use of tests for determining the presence of spore-forming organisms is under discussion, so that water bacteriologists as well as water chemists are active in their researches.

In the field of vital statistics new ideas are also coming to the front. The typhoid fever death rates are becoming so low that they can no longer be regarded as sufficient to measure the healthfulness of a water supply. The so-called Mills-Reincke theorem, which held that for every death from water-borne typhoid fever there were several deaths from other diseases due to water, has been found not to hold generally, although there is much truth in it. Polluted water may cause sickness of one kind or another, which does not find record in the vital statistics of the community. Some more careful measure of the effect of water on the health of the community is urgently needed.

Thus, it is seen that there are activities among engineers, chemists, biologists, and statisticians in the field of water purification, a fact which is full of promise.

RECENT DEVELOPMENTS IN WATER FILTRATION

By Allen Hazen,* M. Am. Soc. C. E.

A Dutch author has grouped all waters in public water supplies in two classes, namely, aggressive and quiet. He defined aggressive waters as those having a tendency to attack and corrode the iron pipes through which they flow and other metal fixtures with which they come in contact. Quiet waters do not have this tendency to attack metals.

Aggressive water is very troublesome in a distribution system. The treatment of water so as to purify it adequately without making it aggressive is one of the most difficult problems at present. Water leaving the filter may be of good color, show a good bacterial test, be free from turbidity and other injurious elements, and may appear to be in all respects suitable for public water supply, but if it is aggressive, it will not be satisfactory to the users. An aggressive water attacks and tuberculates the pipes, reduces their carrying capacity, and will ultimately destroy them. In doing this, however, some of the iron of the pipes is taken up, and it is this iron which makes the water objectionable. The iron taken up causes a supplementary coagulation in the water, and the floc formed in this way settles out where the velocity of flow is low. This deposit is a dirty mud which remains on the bottom of the pipes until the flow is increased sufficiently to move it. It then moves forward with the flow and makes the water dirty and disagreeable, giving just cause for complaint.

There is another condition similar to this, which is frequently associated with it. Water which has received chemical treatment before filtration, frequently passes the filter before the chemical reactions are altogether complete. Such water leaving the filter contains a small quantity of coagulant which separates in the pipes and produces the same conditions as those caused by the iron taken up from the pipes. Probably this condition of incomplete chemical reaction at the time of filtration is found mainly with aggressive waters. At any rate, the two conditions produce similar results and frequently are found together, in which case, the resulting conditions are much worse than they would be otherwise.

The question of why some waters are aggressive and others quiet is presented. Twenty years ago, chemists would have stated that it was, first, a matter of alkalinity, and carbonic acid would have been mentioned as a second important contributing factor. Alkalinity and carbonic acid really have something to do with aggressive qualities, but all the conditions cannot be accounted for by them. Other explanations must be found. Recently, new methods of testing have been originated. The theory of "ions" has been developed, and it is thought by some who have been studying the matter, that a connection between the "ions" and the aggressive qualities of waters can be traced. Tests based on these new ideas have been devised, and have been applied for the first time quite recently, and there now seems to be some hope of a method of ascertaining more definitely when water will be quiet and

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when it will be aggressive. Knowledge comes first, and when engineers understand better why waters are aggressive, it will be easier to find methods of treating them so as to keep them quiet.

A great deal is known in a practical way as to which waters are aggressive and give trouble because of their aggressiveness. The waters of the Great Lakes are almost always quiet and can be filtered and purified in a manner to keep them so. In the Mississippi Valley, from the Alleghanies to the Rockies, the surface waters used for public water supplies are generally hard, and can be handled, by carefully managed treatments, so that the resulting products will be reasonably quiet. Thus, through all that broad expanse of country, the problem of aggressive waters is much less important and difficult.

Soft waters are frequent nearer the coasts, both East and West, and they often carry organic matters of vegetable origin. These waters are much more apt to be aggressive, and it is with them that the greatest practical difficulties have been met. Experience shows that it is difficult to treat some of these waters by methods otherwise effective, which will clear them up and make them satisfactory for public water supplies, and which will leave them quiet. Many of these waters require chemical treatment, and no other means of adequately purifying them has been found, but in some way this chemical treatment stirs up the sleeping ingredients of the water and makes them aggressive.

Decolorizing yellow swamp waters by means of chemicals seems particularly difficult of accomplishment without bringing out the aggressive qualities. The question of color is mainly one of esthetics, and it may fairly be asked whether the increase in aggressive qualities in decolorizing water does not largely or entirely offset the advantage gained, and whether it would not be wiser in some cases to get along with less decolorization.

The chemical treatment of water introduced many years ago had numerous advantages and came to be widely used. Many waters, however, can be treated with entire success without chemical treatment. This depends on the character of the water, and on what is to be removed from it. Chemical treatments may have been used in some cases where it would have been better to have avoided them. Decision to use them may have rested on consideration of all the other factors in the case. When the development of aggressive qualities is also taken into account, perhaps, in some cases, the decision would have been better made the other way.

It may be mentioned that one industrial establishment is now constructing a plant for the purification of water without the use of chemicals. Formerly, chemically treated water was used, and experience with it has indicated the difficulty of furnishing water sufficiently quiet so that it would not damage the delicate products of this particular establishment.

The question of arranging all the treatments of public water supplies so as to keep them always quiet is one of the most interesting and important aspects of water purification.

INTERFERENCE WITH WATER FILTRATION PLANT OPERATION BY WASTES FROM BY-PRODUCT COKE OVENS AND GAS-WORKS

By C. A. Emerson, Jr.,* M. Am. Soc. C. E.

The discharge of liquid wastes from by-product coke ovens and commercial gas plants into streams and lakes from which water supplies are taken, has become, during the past few years, a matter of grave concern to many medium sized and several of the largest cities, due to the disagreeable tastes and odors which these wastes impart to the water.

As the superiority of by-product coke ovens over the old beehive type is now fully established, it follows that the number and capacity of these installations will undoubtedly increase. By-product ovens of necessity must be located reasonably close to a steel works or a large municipality in order to secure a ready market for the gas given off during the coking of the coal, and, therefore, it is almost certain that interference with surface water supplies will also increase, if proper precautions are not taken.

The wastes from the tar separators, the ammonia, benzol, and naphthalene stills of by-product coke ovens, together with the sludge removed from the various gas and oil mains throughout the plant, impart a characteristic taste to river water. This taste may persist for days and, after transportation for many miles, be noticeable in the effluent from well operated, water filtration plants of modern types, taking their raw water supply from the stream. The application of chlorine, even in the minute quantities used for water supply treatment, seems to accentuate rather than diminish these taste-producing characteristics. Under favorable conditions, dilutions as great as several million parts of chlorinated water to one of the wastes have failed to cause complete disappearance of this taste.

Fortunately, the remedy for this pollution lies within the works, and consists simply of evaporation of the contaminated liquid wastes by using them for quenching or cooling the glowing coke taken from the ovens. The waste water which is not turned into a cloud of steam, but passes through the quenching platform, is intercepted in settling basins and again used for quenching. In all the plants coming under the speaker's observation, the water requirements for coke quenching have exceeded the volume of contaminated wastes, so that this remedy has not worked any hardship on the owners.

That the remedy is effective is evidenced by the fact that the largest by-product coke-oven installation in the world is operating on the Monongahela River, ten miles above the intake of a municipal water-works, without causing contamination of the supply. Take another stream in Pennsylvania as an example: The installation of appliances for using liquid wastes from stills for quenching the coke silenced complaints among the consumers of filtered water in several municipalities along the stream about seventy miles below the works. That the method is beyond the experimental stage is indicated by the fact that

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it has been in successful use at numerous plants for several years and that, in some instances, the volume of liquid wastes evaporated is in excess of 1 000 000 gal. daily.

Interference with water supplies by the wastes from plants producing water gas for illuminating and power purposes has been noticeable in different parts of the United States. Definite evidence that the tastes due to these wastes are as persistent, or that they can be detected in as high dilutions, as those from by-product coke ovens, seems to be lacking. There is, however, sufficient evidence to justify the statement that the raw wastes from these plants, when subjected to dilutions available in rivers of considerable size, will cause tastes after transportation by the stream for several miles. With these wastes, also, the taste-producing characteristics are accentuated by chlorination of the drinking water taken from the stream.

Aeration of the raw water, thus contaminated, prior to filtration seems to be of considerable value, but in this case, again, a certain and effective remedy can be developed at the gas-works. It has been found that the installation of settling and skimming basins to permit the removal of the greater portions of the heavier tars and the light floating oils, with the circulation of the basin effluent for re-use in the "gas scrubbers", will not only materially decrease the proportion of active taste-producing constituents, but will reduce the volume of wastes reaching the stream to a small fraction of that which occurred before the water was re-circulated through the plant. In addition, the company recovers marketable materials to offset the cost of operating the circulating pumps.

In one instance coming to the speaker's attention recently, it was deemed advisable to require the gas company to go even further, and to keep all liquid wastes from the stream. This company, by the use of settling basins and circulating pumps, had decreased the volume of wastes from 50 000 to about 5 000 gal. per day. This waste was high in turbidity, had an oily appearance, and was brown in color, but it was found that after it had been strained through a layer of coke breeze, about 6 or 8 in. in thickness, and had received a moderate application of ferrous sulphate solution, it could be passed through a small sand filter at a rate of about 400 000 gal. per acre daily and then become a satisfactory boiler water. Contrary to all expectations, the sand filters did not clog immediately, but were operated for runs of several days' duration between scrapings.

Occasionally, there has been doubt as to legal authority to require abatement of the class of pollutions described, as the laws controlling stream pollution due to the discharge of industrial wastes are weak and vague in many States; however, instances are recorded where applications for restraining injunctions have been granted under the broad general rights of lower riparian ownership and, accordingly, it is believed that in a surprisingly large number of cases definite obligations can be imposed on owners of such works to prevent troublesome stream pollution.

RECENT PROGRESS IN THE REDUCTION OF THE TYPHOID DEATH RATE AND ITS SIGNIFICANCE

By C.-E. A. Winslow,* Esq.

It seems worth while to call attention to the remarkable progess which has been made in the reduction of the typhoid death rate since George A. Johnson, M. Am. Soc. C. E., presented his admirable review of "The Typhoid Toll" before the American Water Works Association, five years ago.† One may go back perhaps even a little farther with profit and take as a basis for comparison the statistics for certain large cities for the period, 1898-1908, presented by Mr. G. R. Taylor.

TABLE 1.—Typhoid Statistics of Cities of the United States, 1898-1908 and 1917-1919.

City and State.	Typhoid death rate per 100 000 population. Average, 1898-1908*.	Typhoid Death Rate per 100 000 Population for 1917, 1918, 1919 (United States Mortality Statistics):		
		1917.	1918.	1919.
Worcester, Mass.	15.3	5.8	3.4	2.8
Fall River, Mass.	15.7	18.3	15.0	3.3
St. Paul, Minn.	16.3	2.6	3.5	3.4
New York, N. Y.	17.6	4.2	3.6	2.2
Rochester, N. Y.	18.5	3.0	1.8	3.4
Jersey City, N. J.	18.5	3.8	5.5	2.4
Newark, N. J.	19.2	4.3	4.5	2.2
Syracuse, N. Y.	19.2	6.1	9.0	7.1
Providence, R. I.	19.5	6.8	5.1	3.8
Milwaukee, Wis.	19.7	6.2	6.3	3.7
Detroit, Mich.	20.9	12.7	7.9	5.2
Paterson, N. J.	20.9	12.8	4.5	3.7
Omaha, Nebr.	23.1	4.4	5.9	5.3
Boston, Mass.	23.3	3.1	2.6	2.4
Chicago, Ill.	25.3	1.9	1.6	1.3
Buffalo, N. Y.	27.0	9.9	7.7	6.6
St. Louis, Mo.	27.7	8.4	7.9	6.6
Scranton, Pa.	28.8	6.6	6.6	1.5
Minneapolis, Minn.	34.9	6.7	8.4	3.2
Baltimore, Md.	35.5	14.4	11.5	8.2
Toledo, Ohio.	35.8	8.0	11.2	4.2
Los Angeles, Cal.	36.8	6.0	3.9	4.8
Memphis, Tenn.	38.7	26.1	16.2	52.8
Indianapolis, Ind.	39.3	10.5	6.6	4.5
New Haven, Conn.	40.5	10.3	4.4	5.6
San Francisco, Cal.	42.0	4.5	4.2	3.0
Kansas City, Mo.	43.0	12.1	15.0	10.9
New Orleans, La.	43.7	23.5	20.0	13.2
Denver, Colo.	48.8	5.7	9.2	3.5
Cincinnati, Ohio.	50.7	4.0	4.8	3.0
Philadelphia, Pa.	53.6	6.3	4.9	4.3
Washington, D. C.	55.5	12.2	11.6	3.7
Louisville, Ky.	57.4	16.3	16.3	10.7
Columbus, Ohio.	57.5	7.6	8.7	3.0
Pittsburgh, Pa.	116.9	12.2	10.7	6.5

* "A Classification and Study of the Typhoid Statistics of the Cities of the United States", by G. R. Taylor.

The statistics for this period are compared with those for 1917, 1918, and 1919, in Table 1, and Table 2 shows the distribution of these cities in the two periods, by their grouping in regard to typhoid mortality. It will be

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† *Journal*, Am. Water Works Assoc., Vol. 3 (1916), p. 249.

noted that in 1898-1908, each of the thirty-five cities had a death rate of more than 15 per 100 000, and that seventeen, or practically one-half of them, had a rate of 30 or more. In the last three years, however, only one city of the same thirty-five has had rates of more than 15, only six have had rates of more than 10, while more than one-third have had rates below 5 per 100 000. This is a truly remarkable achievement and one which is not confined to the cities in question, but which is general throughout the United States.

TABLE 2.—Death Rate from Typhoid Fever in Certain Large Cities of the United States.

Distributed by Classes.

Death rate per 100 000.	Under 5.	5–9.9.	10–14.9.	15–19.9.	20–29.9.	30–39.9.	40–49.9.	50–99.9.	100+
Number of cities in each class, 1898–1908.....	0	0	0	10	8	6	5	5	1
Number of cities in each class, 1917–1919.....	12	17	4	1	1	....	....	....	....

In Table 3 are presented the data for the gradually expanding area of the registration States, which show that for this unit of area the typhoid rate has dropped in thirty years to less than one-third of its 1890 value. The actual accomplishment is even more striking than such a tabulation would indicate for, as the registration area has increased, it has tended to include more and more cities with relatively primitive sanitary organization. Table 4 shows the figures for the registration States as the area existed in 1900, and it will be seen that when the same States are compared, which had a rate of 25.4 in 1900, the rate had fallen to 6.5 in 1916-19, a reduction of nearly 75 per cent.

TABLE 3.—Death Rate per 100 000 Population from Typhoid Fever in the Expanding Registration States for Different Periods, by Cities and by Rural Areas, 1890–1919.

Place.	Period Considered:					
	1890.	1900.	1901–05.	1906–10.	1911–15.	1916–19.
Registration States.....	36.0	25.4	24.9	24.6	16.3	11.9
Cities.....	39.0	25.3	24.5	25.3	14.8*	9.2*
Rural areas	31.4	25.5	25.4	23.8	17.8	13.7

*Municipalities of 10 000 or more inhabitants in 1910.

Such results are highly gratifying, but the speaker would like to utter a word of caution relative to the claims any one may be inclined to make in regard to the factors which have brought about this reduction. One is naturally tempted, when the figures for such cities as Pittsburgh, Pa., are

examined and where the reduction in the typhoid death rate was from 116.9 (1898-1908) to 6.5 (1919), to claim all the credit for the water-works engineer, but a little consideration will show that there are other powerful forces at work.

TABLE 4.—Death Rate per 100 000 Population from Typhoid Fever in the Original Registration States for Different Periods, by Cities and by Rural Areas, 1900–1919.

Place.	Period Considered:		
	1900.	1911–15.	1916–19.
Original Registration States †..	25.4	11.1	6.5
Cities.....	25.3	13.0*	6.9*
Rural areas.....	25.5	8.4	5.8

* Municipalities of 10 000 or more inhabitants in 1910.

† Connecticut, District of Columbia, Maine, Massachusetts, Michigan, New Hampshire, New Jersey, New York, Rhode Island, Vermont.

The data for the expanding registration area presented in Table 3 would suggest that the typhoid death rate has fallen much faster in cities than in rural areas, but this conclusion is wholly fallacious, for the expanding registration area has tended to increase the proportion of sub-standard rural areas much faster than the proportion of sub-standard urban areas. If the rural and urban changes in the death rate in the original registration States of 1900 are compared, it will be found that the reverse condition obtains; in other words, when the same group of cities is considered, the rural death rate has decreased more markedly than the death rate in urban communities. This fact alone is sufficient to indicate the complex nature of the problem, since improvements in water supplies have certainly been vastly more effective in cities than in rural districts. The improvement in privy sanitation, the war against the fly, the pasteurization of milk, and, since the World War, the use of typhoid vaccine, have each played an important part in the conquest of typhoid fever, and there is glory enough for all in the fact that at last the typhoid statistics of the United States can be compared with those of Europe without the sensation of humiliation.

THE EFFECT OF WATER PURIFICATION AND IMPROVEMENTS IN WATER SUPPLIES ON THE TYPHOID FEVER DEATH RATE IN NEW YORK STATE

By C. A. Holmquist,* Esq.

Before discussing the reduction of typhoid fever, due to improvements in water supplies, the speaker will review briefly the part played by the New York State Department of Health, more particularly the Division of Sanitary Engineering, now the Division of Sanitation, in improving the public water supplies of the State through its general supervision over their sanitary quality. The first State Board of Health of New York was organized in 1880, under the provisions of the Public Health Law. This Board consisted of nine members, nearly all of whom were physicians.

In 1901, the State Board of Health was abolished, and a State Department of Health, with a commissioner at its head, was created. In 1906, the Division of Sanitary Engineering was established. Up to that time, relatively little attention had been paid to public water supplies. Engineers were employed from time to time to advise the Board and to investigate matters pertaining to sewerage and sewage disposal, stream pollution, and other sanitary engineering problems, and although rules and regulations had been enacted for the protection of certain surface supplies from contamination, no comprehensive investigations had been made to determine the conditions of the water supplies in the State, or to improve them.

Since the establishment of the Division of Sanitary Engineering in 1906, one of the principal, if not the most important, functions of the Division has been the supervision over the sanitary quality of public water supplies. Of about 550 public water supplies in the State all have been investigated and reported on at least once and many of them a number of times.

Rules and regulations for the protection of about ninety surface supplies have also been enacted. Since 1906, the number of persons served by public water supplies in New York State has increased from about 6 000 000 to 8 500 000, and the number of persons supplied with purified or treated water has increased from 700 000 to nearly 7 000 000.

The results of these improvements are clearly reflected in a decrease of the typhoid fever death rate in the State as a whole. Before 1906, the average death rate from typhoid fever for the whole State, including New York City, was 23.4. Since 1906, the average rate has been 9.4, a reduction of 14 per 100 000. It will be noted also that the rate has decreased each year with marked regularity, until 1920 a remarkably low level of 3.5 was reached, a figure which about 20 years ago was generally considered to be the probable irreducible minimum for the United States. In all probability, the rate will be lower, but the decrease necessarily will be less marked in the future than in the past. The figures available up to the end of September, 1921, indicate that the rate for this year will be slightly less than those for 1920.

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Although other factors, such as improvements in milk supplies, etc., no doubt have had some effect in decreasing the typhoid fever death rate, the speaker believes that the decrease has been due largely to improvements in water supplies.

The municipal authorities of Cohoes, N. Y., constructed filters and commenced sterilizing the water supply in 1911. Between 1900 and 1911, the average death rate from typhoid fever was 85.5, fluctuating from a minimum of about 58 to a maximum of about 133. Since the filters were built, the average death rate has been 17.3 and, for two years, no deaths from typhoid occurred in that city.

Filtration and sterilization plants were constructed in Niagara Falls, N. Y., in 1911. Before that time, the average death rate from typhoid fever was 131.8; since the establishment of filters the average death rate has been 9.4.

Albany had an average typhoid fever death rate of 89.4 per 100 000 before the construction of its filter plant in 1899. Since the filters were built, the average death rate from typhoid fever has been 17.5, and it has been gradually decreasing until in 1920 the rate was about 3 per 100 000.

At Binghamton, N. Y., the average death rate from typhoid before the filters were built, was 56.2, and the average death rate from typhoid since the construction of the filters has been 11.7.

It is evident that in each case a decided drop in the typhoid fever death rate occurred immediately after the construction of the filtration plants. It is also evident that the city, the water supply of which was most grossly polluted before purification, had the highest typhoid rate and the one the water supply of which received the least pollution had the lowest rate. The distance which the pollution has to travel and the velocity of the stream are, of course, important factors in the case of water supplies derived from rivers.

For Niagara Falls, which had the highest death rate of any city in the State before the construction of filters, the water supply is taken from the Niagara River. Buffalo, N. Y., with a population of about 500 000, discharges its untreated sewage into the river about 15 miles above the intake of the Niagara Falls water supply, and the raw sewage from the Cities of Tonawanda and North Tonawanda, N. Y., with a total population of 25 000, is discharged into the river about 10 miles above Niagara Falls. With the sewage of 500 000 people discharging into a comparatively swift stream at a relatively short distance above the water supply intake, it is not surprising that Niagara Falls had a high typhoid fever rate before water purification was provided.

The typhoid death rates at Cohoes and Albany, before the construction of filters were nearly the same, and the raw water is of a somewhat similar character. Cohoes obtains its water from the Mohawk River about 18 miles below where the sewage from Schenectady, with a population of about 90 000, is discharged into the river, formerly without any treatment whatever.

Albany obtains its water supply from the Hudson River about 8 miles below the mouth of the Mohawk River, and the intake is from 4 to 10 miles below Troy, Cohoes, and Watervliet, which have a combined population of

110 000. The sewage from all these cities is discharged directly into the Hudson River without treatment.

The Susquehanna River, from which the City of Binghamton obtains its water supply, receives comparatively little raw sewage within 50 miles of the intake. The typhoid fever death rate at Binghamton, although high, was nevertheless much lower than at either of the other cities referred to.

In closing, the speaker wishes to point out that to Theodore Horton, M. Am. Soc. C. E., who was Chief Engineer of the State Department of Health from 1906 to July 1st, 1921, is due in a large measure the credit for the improvement of public water supplies in New York State.

PURIFICATION OF SOFT COLORED WATERS

By Robert Spurr Weston,* M. Am. Soc. C. E.

Engineers engaged in water supply work are in substantial agreement that a drinking water, to satisfy critical modern consumers, should have an average color of not more than 10 parts per million on the platinum scale, and its maximum color should not exceed 15 parts. Supplies from ground sources and from many lakes and ponds conform to this standard, but most of the clearer surface waters possess a brownish coloration, well above the limit of 15 demanded by the modern consumer. In Massachusetts, according to the reports of the State Department of Health, practically all the ground-water supplies which are free from iron, have colors not exceeding 10, while of 86 surface supplies, 50 or 58% have average colors in excess of this standard. The appearance of these colored waters draws unfavorable comment from Western visitors who are accustomed to water filtered clear and colorless, from turbid streams.

A growing appreciation of the esthetic characteristics of drinking water is leading to the betterment of these colored supplies by purification, the current methods being storage and filtration. Storage for months or years in good reservoirs effects a decolorization of water. In this, iron plays an important part in flocculating minute particles of suspended matter which constitute what is called color. Several examples of this were given by Ralph H. Stearns,† Assoc. M. Am. Soc. C. E., which show reductions of from 4 to 69% through storage. It is most often impracticable to store waters long enough to decolorize them sufficiently to compare them, with a reasonable degree of favor, with filtered waters and the average ground water, and, for this reason, as well as further to safeguard the public health, many stored colored waters are filtered. This practice is usual abroad, and is coming into increasing favor here, as evidenced by the new purification plant at Hartford, Conn., and the plans for the additional supplies for New York City, and Providence, R. I. The first large filters were built at Cleveland, Ohio, and Somerville, N. J., and the second one at Washington, D. C. The filters at Hartford are of the slow type, and although the color of the effluent will probably not average as low as 10 parts per million, a supply of reasonably good appearance is assured by this method. Of course, color removal by slow filters has been the subject of many discussions, and the statements made have rarely taken into account the physical and chemical nature of color.

Most of the color in water is in the form of a colloidal suspension, that is, the particles are so fine and so dispersed in water that they remain in suspension and act like true solutions, except that the Tyndall ray and the ultra-microscope reveal them. These colloidal particles are not all alike. They possess electrical properties, and when electrolyzed some of the particles will migrate to the positive, some to the negative pole. A small part of the color is apparently in true solution and is not affected by the electric current. It is reasonable to believe that colored water from a large lake or reservoir

* Cons. Engr. (Weston and Sampson), Boston, Mass.

† *Journal*, New England Water Works Assoc., Vol. 30, p. 20.

will contain finer particles of color than a river water. This belief is supported by the higher color removal by sand filters supplied by river than by reservoir waters. A slow sand filter will remove one-third of the color, but when treating certain river waters such filters may remove more than one-half the color, although they are usually unable to remove more than one-fifth of the color from a stored water.

There are waters, therefore, which require chemical treatment prior to filtration. In nearly all such cases, sulphate of alumina with or without an alkali has been used. The dosing has been based on the appearance and alkalinity of the water, and the results, especially in small plants operating without laboratory control, leave much to be desired. Recently, George C. Whipple, M. Am. Soc. C. E., has stated:

"The use of alum with short periods of coagulation and mechanical filtration of the ordinary type is, in my opinion, inappropriate to a soft colored Massachusetts surface water. * * * * The corrosion problem in our State is serious and must not be made more so by inappropriate chemical treatment."

The mechanical filter, *per se,* is an efficient device, and the remedy lies in better chemical treatment. If the waters could be treated more scientifically, the period of coagulation might be reduced and the rate of filtration increased. Better treatment, however, must await the application of modern chemical and physical theories to practice, and this will require much investigation and many trials before any rules can be written for the non-technical operator.

At present, sulphate of alumina is added until coagulation takes place, and, in most cases, soda is added to maintain an alkalinity of at least 7 parts per million in the water delivered to the mains. Frequently, this method fails. At some places, such as Warren, R. I., soda cannot be used at certain times of the year, and the alkalinity is maintained by the addition of calcium carbonate in the form of powdered chalk. If sulphate of alumina is added to excess, better decolorization will be obtained, but the treated water will corrode the distribution system. In the one case, the soda has been added to the filtered water. At Wilmington, N. C., the water is first overdosed with alum, and after it has passed through the coagulating basin, it is dosed with enough soda or lime to insure a final alkalinity of 10. In these cases, both the optimum decolorization by sulphate of alumina and the inhibition of corrosive action is attained. At Exeter, N. H., it was impossible, even by adding sulphate of alumina to excess, to reduce the average color much below 20 parts per million. Apparently, the water contained a large quantity of positively charged coloring matter which was not affected by aluminum hydrate having the same charge. By pre-treating the water with 0.5 parts of chlorine per million, a condition was brought about under which treatment with a small dose of sulphate of alumina easily reduces the color below 10 parts per million.

Ordinarily, one believes that if an equivalent quantity of soda is added to a unit dose of sulphate of alumina, aluminum hydrate will be produced. Try this in some cases in practice, and what results—a re-solution of the color.

In some cases, reduce the equivalent dose of soda to, say, one-half, and a good decolorization may result. These facts are known, but the principles on which they are based are not yet clear.

There is also the problem of after-precipitation. In at least four rapid filter plants supplying New England towns, some of the aluminum hydrate passes the filters in a colloidal form to coagulate later in the distribution system. This was the chief of the many objections which the late Hiram F. Mills, Hon. M. Am. Soc. C. E., raised against alum treatment. This fault is noticed even at plants with coagulating basins of relatively large capacities, although not so frequently as at plants where the chemicals are applied directly to the influents of pressure filters. The trouble occurs most frequently in cold weather when all chemical reactions are retarded.

From what has been said, it is obvious that water purification experts are trying to solve what is, perhaps, the most difficult water purification problem, without a knowledge of the factors which enter into it, and until they can elucidate these factors, the demonstration of the theories is impossible.

For guides, engineers have depended largely on the color and alkalinity determinations. What do these mean? Simply a measure of the intensity of the one and the combining power of the other. No regular determinations record the nature of the color, and although the brown of one water may appear to be similar to the brown of another, it is no sign that they will respond to the same treatment. Thorndike Saville, Assoc. M. Am. Soc. C. E.,* showed how varied were the components of color in water. His experiments demonstrated the ultra-microscopic character of the "suspensoids" and "emulsoids", to use the modern terms, which make up the bulk of color in water, and that these carry electrostatic charges, positive or negative, depending on, and varying with, the water. Usually, the color particles have a negative charge.

It is obvious that the difficulty of finding the charge of the color particles and the necessary treatment is not much facilitated by the ordinary water analysis. In practice, it is assumed that the color carries a negative charge, and aluminum hydrate, with its positive charge, is used to neutralize and coagulate it. This treatment does not dispose of the positively charged color particles, except that these may be removed by the absorptive action of the precipitate produced by the negatively charged particles and the hydrate. Special treatment for these positively charged particles is necessary, and the substances added must not neutralize the effect of the positive aluminum hydrate. The good effect of chlorine at Exeter, and of carbon dioxide in the experiment of Mr. M. C. Whipple† in increasing the electro-negative character of the color and, consequently, the decolorizing power of a given dose of sulphate of alumina, leads one to hope that methods for determining the electrostatic charges of color particles will come into more common use and their results will indicate more rational treatment than that commonly used at the present time.

* *Journal*, New England Water Works Assoc., Vol. 31, p. 78, *et seq.*

† *Loc. cit.*, p. 116.

The alkalinity determination simply expresses the quantity of sulphuric acid in terms of calcium carbonate required to neutralize the water. It does little else. Practically all the colored waters supplied to towns are alkaline. Yet, how different are the treatments required by waters of about the same alkalinity and color. An equivalent of magnesium hydrate has the same combining power as an equivalent of caustic soda. The former is called a weak, the latter a strong, alkali, and these adjectives describe their effects. By determining the hydrogen-ion concentration, these differences of potency can be measured. This determination is coming into use; it has already been made part of the daily routine at Baltimore, Md., and other places. Apparently, there is an optimum hydrogen-ion concentration for each water, where coagulation is most nearly complete. It is not far from PH = 7.0. The hydrogen-ion determination is not difficult, and the speaker believes that it may show what conditions of the dissolved salts in water, that is, the electrolytes, are most favorable for flocculation of the colloidal color, probably in many cases by suggesting treatment with other chemicals as well as with sulphate of alumina.

The problem, therefore, is to substitute a specific for a panacea treatment by a more accurate analysis of color and a more proper dosage with chemicals, also by the creation of the most favorable condition in the solution—the water —for the flocculation and precipitation of the neutralized colloidal color. This seems to be complicated, but the speaker believes it will be worked out to a satisfactory solution in practice. Indeed, the collection and study of the more refined electro-chemical data should explain the successes and failures in practice, and should help to rationalize the treatment of colored waters. On the practical side is the lure of a greatly reduced cost for chemicals, which should stimulate research in this important field.

THE OPERATION OF RESERVOIRS FOR WATER SUPPLY

By Samuel A. Greeley,* M. Am. Soc. C. E.

The developments of storage for water supply throughout many parts of the Upper Mississippi Valley is frequently a relatively costly project. The river valleys are wide and flat, and the slope of the streams is moderate. Thus, at Decatur, Ill., to impound about 9 000 000 000 gal. to furnish an estimated minimum yield of 42 000 000 gal. per day, it is necessary to flood about 5.5 sq. miles and the total cost of the project is almost $2 000 000. Several other water-impounding projects which have come to the speaker's attention, have called for expenditures in excess of bond limitations and have been financed by popular subscription.

Furthermore, south of Wisconsin and Minnesota, ponds, lakes, and streams for recreation purposes are scarce and inaccessible and by no means as plentiful as they are throughout New England, for instance, and parts of New York State and New Jersey. For these reasons, among others, there is a strong disposition to permit the use of these reservoirs for recreation purposes, including bathing, boating, fishing, and the like. The most extreme case which has come to the speaker's attention is Lake Milton, in Ohio, which furnishes the water supply for Youngstown, Warren, Niles, and other small communities in Ohio. This reservoir covers about 2.65 sq. miles and impounds about 9 000 000 000 gal. There are a number of resorts about its shores, and bathing, boating, fishing, and picnicing are allowed. Below the dam, the water flows several miles in open channel to the intake of the filter plant, receiving pollution on the way, which masks the effect of reservoir use.

It should be noted especially, however, that many of the surface water supplies of the Upper Mississippi Valley are turbid and otherwise unsatisfactory without filtration, so that modern water filtration plants are, in general, in operation in the case of the larger projects. This circumstance tends to render officials and the public less sensitive to the possibilities of pollution at the source.

Earlier Discussions.—The use and maintenance of reservoirs has been under consideration from time to time since 1907, when a thorough discussion† was presented before the Society on the relative value of the control of drainage areas and the installation of water filtration plants for the protection of public water supplies. The discussion at that time, however, did not relate primarily to the use of reservoirs as this use affected a safe load on water filtration plants.

A later, and a very able, discussion was presented in 1920, by X. H. Goodnough, M. Am. Soc. C. E., before the New England Water Works Association, deprecating the use of reservoirs for fishing and boating and instancing a number of cases where permission to fish resulted adversely. In most of these cases, however, the relation of water filtration was not stated.

In a still further discussion,‡ the following conclusions were offered by George A. Johnson, M. Am. Soc. C. E.:

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* Hydr. and San. Engr., Chicago, Ill.

† *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 367.

‡ *Journal*, Am. Water Works Assoc., July, 1921.

(*a*).—Maintain the catchment area in as sanitary a condition as practicable; that is, guard against gross pollution entering the streams and lakes which drain the water-shed.

(*b*).—Store the water in natural lakes or artificial reservoirs, provided such storage is available or dictated by sound engineering principles.

(*c*).—Coagulate and filter.

(*d*).—Sterilize.

These comments, however, are not specific as to the use and protection of reservoir supplies.

Apparently, this matter is not finally settled in all cases, as indicated by a recent Act submitted to the Massachusetts Legislature relative to the use of Lake Cochituate in the Town of Natick for boating and fishing. The officers of the New England Water Works Association have protested against the passage of this bill.* Efforts should be directed to (1) the control of human activities to prevent undue pollution; and (2) the control of natural processes to prevent deterioration of physical qualities.

Principal Considerations.—The operation of reservoirs for water supply may be discussed from two principal considerations: (*a*) the use of reservoirs for other than water supply purposes; and (*b*) their maintenance for water supply purposes.

The first consideration includes the regulation of the reservoir for bathing, boating, fishing, picnicing, and the development of the shores into camps, resorts, and residential and industrial districts. The effect of these various uses on the condition of the water as affecting the safe load on water filtration plants is the present-day criterion.

The second consideration includes the necessary operations for the maintenance of the reservoir, such as the control of objectionable growth of microscopic organisms, the proper draining and clearing of the banks, the protection of the shores, the regulation of storage, the handling of silt, the control of fish life, the drainage of swamps, the control of erosion, and the like.

In connection with the approaching completion of the impounding reservoir at Decatur, Ill., the speaker has gathered some data indicating practice along these general lines in a number of other situations. (See Tables 5 and 6.)

Summary of Data on the Use of Impounding Reservoirs.—Table 5 shows the practice with filtered water supplies. In general, it is indicated that no bathing is allowed, with the exception of Youngstown, Ohio, and at Fort Worth, Tex., some bathing is permitted. There is indicated a more general permission for boating, as this is forbidden entirely in only seven towns out of the fourteen records available. Fishing from the shores is almost generally permitted, and quite generally also from boats, although this latter practice is commonly regulated through permits or licenses. Picnicing and camping seems also to be allowed, with the establishment of camps and resorts permitted in about one-half the cases. Some method of patrol is almost always undertaken, and among the more comprehensive of these methods are daily inspections by five mounted patrolmen over a drainage area of 110 sq. miles for the Oakland, Cal., water supply.

* *Journal*, New England Water Works Assoc., September, 1921.

Table 6 indicates the data for unfiltered water supplies. In only one case is bathing reported as permissible, that is, at Paris, Ill., in which city the water is not used for drinking. Boating also is forbidden, whereas fishing from the shores is forbidden in seven cases out of eleven. Fishing from boats is also generally forbidden, and this is also true of picnicing, camping, and the development of resorts.

In the case of the impounded or lake supplies for Boston, Mass., and New York City, very careful control is exercised, fishing and some boating is allowed by permit, and some bathing occurs near some of the New York reservoirs.

Practice thus indicates a somewhat freer use of reservoir waters in the case of filtered supplies.

Comments on Bathing.—The matter of bathing appears to be the most important consideration. It should be considered not only for its effect on the quality of the water in the reservoir, but also for its effect on the bathers themselves. The effect of crowded bathing beaches on the bathers has recently called for the serious attention of the Health Departments in Chicago, Ill., Milwaukee, Wis., and elsewhere along the Great Lakes. A suggestion is offered that better bathing facilities can be provided in specially designed pools or ponds, in which the circulation and purification of the water can be controlled. With present-day understanding of the quite general distribution of typhoid carriers, the regulation of bathing assumes increasing importance. For a short time in 1921, several public bathing beaches in Chicago were closed.

Maintenance of Reservoirs.—No large impounding reservoir for water supply can be left to itself, and some more or less continuous maintenance is required. In the first place, developments on the drainage area above the reservoir should be convassed frequently so that gross pollution may be avoided. In the second place, the development of undesirable microscopic growths must be watched and controlled. Methods of control include a reasonable removal of the causes of such growths, the destruction of such growths by the application of minute quantities of copper sulphate, and the improvement of the water by aeration before filtration.

The third consideration involves the maintenance of satisfactory shore conditions, which can often be accomplished by draining, clearing, and burning over marginal areas during low water. It is believed to be particularly important that constant patroling should be undertaken, probably by boat and automobile, so that the establishment of gross nuisances may be prevented at all times, as well as minor infringements of the reservoir use.

Recent English Data.—The speaker's attention has been called to a moderate use of impounding reservoirs for fishing in England. A book by Ernest Phillips, entitled "Trout in Lakes and Reservoirs", has been published, which offers some interesting data. It appears that within the last few years about forty or more English cities have stocked their city water supply reservoirs with fish and opened them to the public. In some of these cities, fishing tickets are issued, which cost at Manchester 25 cents per day and at Huddersfield 50 cents per fishing day. The speaker is not informed as to the extent of water filtration at these supplies, nor as to which, if any, are

TABLE 5.—Water Supply Data: Use of Impounding Reservoirs for Filtered Water Supplies.

City.	Reservoir Data:		Use of Reservoir:						Method of patroling.
	Capacity, in million gallons.	Area, in acres.	Bathing.	Boating.	Fishing from shores.	Fishing from boats.	Picnicing and camping.	Resorts.	
Akron, Ohio	2 400	769	No	No	Yes	Some	Some	No	One man.
Columbus, Ohio	1 720	363	No	No	Yes	No	Yes	No	Two river patrols.
Dallas, Tex.*	.....	.....	No	Yes	Yes	Yes	Yes	No	
Fort Worth, Tex	2 145	.....	Yes	Yes	Yes	Yes	.....	.....	Sanitary patrol.
Girard Water Company, Pa	.....	.....	No	No	Yes*	No	No	No	Caretaker at each reservoir.
Jacksonville, Ill	453	265	No	Yes	Yes	Yes	Yes	No	Under Park Board.
Mt. Vernon, Ill	350	.....	No	Yes	Yes	Yes	Yes	Cottages	Some patroling.
Oklahoma, Okla	8 000	2 700	No	No	Yes	Yes	Yes	No	None.
Oakland, Cal	19 155	1 310	No	No	No	No	No	No	Five mounted patrols.
Ravenna, Ind	.....	85	No	Yes	Yes	Yes	Yes	Cottages	Shores treacherous.
Reading, Pa	150	31	No	Some	Yes	Yes	Yes	Caretakers	Monthly.
Springfield, Mass	2 500	214	No	No	No	No	Little	No	Caretaker resident.
Wilkes-Barre, Pa	9 000	.....	No	No	†	No	No	No	Daily patrol.
Youngstown, Ohio	9 000	1 700	Yes	Yes	Yes	Yes	Yes	Yes	None.

* Subject to general regulations.
† Limited fishing from shore permitted when water is filtered.

TABLE 6.—Water Supply Data: Use of Impounding Reservoirs for Unfiltered Water Supplies.

City.	Reservoir Data:		Use of Reservoir:						Method of patroling.
	Capacity, in million gallons.	Area, in acres.	Bathing.	Boating.	Fishing from shores.	Fishing from boats.	Picnicing and camping.	Resorts.	
Altoona, Pa	1 030	130	No	No	No	No	No	No	Daily patrol.
Bellevue, Ohio	250	.....	No	No	Yes	No	No	No	Patrolmen, May to November.
Centralia, Ill	850	.....	No	Yes	Yes	Yes	Yes	Yes	None.
New London, Conn	600	225	No	No	No	No	No	No	Regular trips.
Omaha, Nebr	100	.....	No	No	No	No	No	No	Watchman.
Paris, Ill.*	300	100	Yes	Yes	Yes	Yes	Yes	Yes	None.
Portland, Ore	(?)	500	No	No	No	No	No	No	U. S. Forest Service.
Rochester, N. Y	7 328	2 476	No	No	No	No	No	No	Constant inspection.
San Francisco, Cal	.....	.....	No	No	No	No	No	No	Seven mountain men.
Seattle, Wash	270	.....	No	No	No	No	No	No	Fenced in.
Syracuse, N. Y	Skaneateles Lake		No	No	Yes	Yes	Yes	Cottages	Patroled by water and road.

* Water not used for drinking.

chlorinated. It is the general understanding in the United States that few of them are filtered. An interesting account of these supplies appears in the *Outlook* for October 12th, 1921.

Rules and Regulations.—A number of rules and regulations governing the use of reservoirs have been published and others have been suggested. Among the most complete are those of the Massachusetts State Board of Health, the Department of Water Supply, Gas and Electricity of New York City, a suggested law by W. H. Dittoe, M. Am. Soc. C. E., Chief Engineer of the Ohio State Board of Health, and ordinances recently passed at Dallas, Tex., and elsewhere. The aim of these rules and regulations is to prevent entirely the use of the reservoir for fishing, bathing, boating, and the like, or to permit some boating and fishing under careful regulation through permits and licenses. Bathing appears to be quite generally denied.

Summary and Conclusions.—The brief time available for the presentation of this subject of the operation of reservoirs for water supplies, and the multitude of differing local conditions, do not permit general definite conclusions. Practice and experience indicate that bathing, except under most favorable conditions, should not be permitted, and that consideration should be given to the protection of bathers as well as to the protection of the water supply. With unfiltered supplies, the tendency is frequently against boating, fishing, and the like. In the case of filtered supplies, the use of reservoirs for such purposes appears to be increasing under the most thorough and careful sanitary patrol.

DISCUSSION

LOUIS L. TRIBUS,* M. AM. SOC. C. E.—For a great many years the speaker's firm had under its observation and care a Western water-works system, the supply for which came from a flashy river, varying greatly in alkalinity and turbidity.

During different seasons and storm conditions, the alkalinity ranges from 40 to 320 parts per million and the turbidity, from zero to 5 000 (5 000 marks the limit on the gauge); how much worse it has been is not known. The point of interest, however, was the treatment of the water with coagulant. Taking the average through the year, 1½ to 2½ grains of sulphate of alumina were required per gallon, and, at times, it was necessary to increase the dose greatly. On one occasion, when the turbidity was at the maximum, the water would not yield until the alum dose was increased to 20 grains. Then, it suddenly yielded and although the conditions seemed to be the same, the alum dose was decreased at once to 11 and then to 7 grains, with perfect results.

Another fact of special interest is that an advantage was gained by dosing the raw water at different stages in its passage. From the river, the water first passed through four successive sections of a sedimentation or coagulation basin; and then to mechanical gravity open filters. Alum was first added at the point where the centrifugal pump took the water for discharge into the first basin, and since the water entered near the bottom of that first basin, a second dose could be given. The flow passed through a port about 3 ft. from the bottom of the wall, between the first and second chamber, and, from the second to the third section, it passed at a little higher elevation. At this port a third dose of alum was added. The water then passed through the third basin into the fourth, and through that and a short pipe line to the filters. The final dose of alum was given just before the water reached the filters. It was found that by using these successive proportional charges, far better results could be obtained than by using the full dose at any one point. The clearest water was neither near the bottom nor at the top, but from about 6 in. to 1 ft. below the top.

G. F. CATLETT,† ASSOC. M. AM. SOC. C. E.—The discussion on colored waters by Mr. Weston is interesting, particularly his reference to such waters in Eastern North Carolina. Considerable study has been, and is being, given to the treatment of these waters as they occur in that State and some interesting results have been published regarding the treatment of the colored water at Wilmington, N. C.

The outstanding facts brought out in connection with these studies, are that the treatment of colored water requires close and scientific regulation of chemicals and of the general treatment, and that it does not require as long a retention time in the coagulating basins and probably not as low a velocity as is required by turbid waters.

* Cons. Engr. (Tribus and Massa), New York City.

† Prin. Asst. Engr., North Carolina State Board of Health, Raleigh, N. C.

In the case of the colored waters of North Carolina, and it is true in general of those found elsewhere, the alkalinity is rather low, and additional alkali must be applied. In applying this alkali a careful adjustment of the ratio of aluminum sulphate to alkali was found to be necessary, so that the hydrogen-ion concentration will be close to the iso-electric point. If this is done, coagulation will occur much more quickly than in the case of turbid, clay waters, and the floc will be in larger, heavier particles. If slightly more alkali is added than is needed to secure the iso-electric point, coagulation will occur in such small particles that no satisfactory sedimentation or filter efficiency can occur. It is obvious that this required control must be under complete laboratory test and under the supervision of an operator thoroughly conversant with the principles involved. In order to secure a factor of safety against free alum going over to the filters, it is found necessary to add a small quantity of alkali as a secondary treatment after coagulation and sedimentation. The quicker coagulation secured under proper treatment and the larger and heavier floc are the reasons for suggesting a lower basin retention time and a higher permissible velocity in the basin.

In treating the colored water at Wilmington, where complete laboratory tests and scientific control is provided, satisfactory color removal is secured with the economical use of chemicals. At Elizabeth City and Lumberton, where there is no trained technical supervision, rather unsatisfactory results are secured, with a much higher consumption of chemicals.

The reference to the brackish water at Wilmington is entirely aside from the difficulty encountered in treating a colored water. Following an abnormal dry spell, such as has been experienced this year (1921), in conjunction with certain wind and other conditions in the ocean, sea water is backed up thirty miles distant to the intake. The October, 1921, report from the Wilmington plant shows chloride content as high as 1 600 parts per million. The same trouble is experienced to a greater or less extent at other towns along the coast. As the condition exists for a short period only, the remedy is raw-water storage sufficient to carry over the duration of salt water.

It is of interest that this condition at Wilmington, in its more aggravated form, was coincident with the construction of locks and dam on the river above the intake.

JOHN R. BAYLIS,* ASSOC. M. AM. SOC. C. E.—Hydrogen ions having been mentioned by Messrs. Whipple, Hazen, and Weston, the speaker will discuss that subject briefly. The laboratory at the Montebello Filters, of the Baltimore City Water Department, was one of the first to use hydrogen ion in plant control and it is now one of the most important tests. The water at Baltimore is comparatively soft, the alkalinity being about 40 and the total hardness about 50. It is corrosive before treatment, and after the application of alum, unless neutralized by an alkali, it is so corrosive that when it reaches the consumer it may look about as turbid as it did before filtration, due to corrosion of the pipes. It is necessary, therefore, to reduce its cor-

* Prin. San. Chemist, Baltimore City Water Dept., Montebello Filters, Baltimore, Md.

rosive qualities, which is now done by adding lime near the outlet of the mixing basin.

Mixing basins are a feature frequently overlooked in filter design, and it may be well to discuss briefly this part of the plant which helps bring about the chemical reactions. Some years ago, the speaker had the experience of trying to operate a plant which had no mixing basin, and after one year of unsatisfactory efforts, a basin was constructed, and immediately the desired results were obtained. Some interesting experiments have been made in California on mechanical agitation to produce the desired mixing, and the speaker believes there is something in that method, because in Baltimore, several years ago, when iron and lime were used as a coagulant, good coagulation was frequently hard to obtain. It required about ½ hour for the water to pass through the mixing basin, which is longer than in most plants. Experiments showed that when this water, which had received the ½-hour mix without producing a satisfactory coagulation, was stirred slowly for 20 or 25 min., excellent results were produced. In recent experiments on mechanical agitation, the speaker has found the time element important, as well as the saving in chemicals. A turbidity requiring about ¾ grain per gal. of alum and ½ hour in the mixing basin of the plant, could be properly coagulated with ⅔ or ½ grains per gallon, if stirred for 25 or 30 min. with the experimental apparatus.

It has been stated that coagulation with colored waters is difficult, but the precipitation is very rapid when once obtained. The chances are that this colored water was overdosed with chemicals, which produces rapid precipitation after the floc is formed. A longer period of mixing may change the results with considerable saving in chemicals. The speaker favors large coagulating basins even though there may be rapid precipitation. If the total capacity is not needed, longer periods between cleaning may be allowed. From experiences in Baltimore, larger basins are preferred. Chemical reactions are sometimes slow, and, at times, it seems that such reactions are not complete until after the water has passed the filters. The capacity of the basins, however, does not always give an indication of the time in passing through, as float tests have shown that, in less than 1 hour, part of the water may pass through basins having capacities of 2½ or 3 hours.

It is possible to prevent corrosion almost entirely by adjusting chemical treatment to the proper hydrogen-ion concentration. With soft waters this may mean a PH value of nearly 9.0 and with hard waters the value may be as low as 7.0, or even less, without producing corrosion. For most waters, in adjusting the hydrogen-ion concentration when alum is used, the alkali should be added after filtration. Preparations are now being made at Baltimore to add the lime after filtration. The water before treatment has a PH value of about 7.0, which is reduced to about 6.5, or less, by the addition of alum, and it is later adjusted to about 8.4 or 8.6 by the addition of lime. There seems to be nearly complete precipitation of the alum if the PH value is not below 6.5, but beyond that it may not be complete. On account of the low alkalinity in some waters, it may be necessary to add some of the alkali before the alum, but only enough should be added to produce a PH value of about 6.5 after application of the alum, and then adjust it to the proper

hydrogen-ion concentration after filtration. In Baltimore, it is possible to re-dissolve about one-half the aluminum hydrate by applying lime in doses slightly higher than are necessary to the coagulated water. These re-dissolved compounds may have no effect on the water, but chemicals are wasted by this practice.

The hydrogen-ion concentration is one of the easiest determinations to make, and possibly is the only test that will enable the filter operator to be sure of the corrosive qualities of the water treated. As soon as the proper concentration is determined for a certain water, the necessary adjustments can be made, and complaints about corrosion or red water will cease. The speaker believes that the hydrogen-ion determination is one of the most important tests.

P. H. Norcross,* M. Am. Soc. C. E.—In the broad field of mechanical filtration, several types of filters which might have been referred to, have not been mentioned by Mr. Whipple in his discussion, namely, the drifting sand filters which are operated at Toronto, Ont., Canada, and filters of the pressure type, which are used in Atlanta, Ga. The filters at Toronto are operated successfully at a capacity about 40% higher than the usual gravity type. In Atlanta, there is located the largest filter plant of the pressure type in the United States. It has a rated capacity of 3 000 000 gal. of the old Hyatt type and of 18 000 000 gal. daily of the Continental Jewel pressure type. These filters are operated as gravity filters under about 30 ft. of head, and during the season of peak demand, have operated successfully at 100% overload. This discussion is not a brief in favor of this type of filter plant, but indicates its flexibility. The City of Atlanta is doubling its filter capacity, the program contemplating 21 000 000 gal. daily of the open gravity filter type.

In speaking of filtration Mr. Hazen referred to the character of water to be treated. On the eastern seaboard of the Appalachian and Blue Ridge Mountains, there are two main classes of water to be considered. The larger of the two classes are surface streams which carry heavy turbid red clays, and are located principally above the coastal plain. Below the coastal plain and adjacent to tide-waters, the usual plan of securing a municipal water supply is from deep wells. Pensacola and Jacksonville, Fla., Savannah, Ga., and others are using water of this character successfully.

Little trouble is experienced in the proper coagulation of water in the Appalachian and Blue Ridge sections, for during the greater part of the time there is sufficient alkalinity. The main difficulty is during certain seasons when there is a lack of alkalinity which must be provided artificially. In his comments, Mr. Weston has mentioned Wilmington, N. C. A rather interesting sidelight on the water supply of Wilmington developed during the summer of 1921. This city draws its water supply from the east fork of the Cape Fear River, the flow of which is much less than that of the western branch, the confluence of the two occurring at Wilmington. During the recent drought, the tidal water backed up beyond the intake of the city plant to such an extent that the supply contained approximately 1 200 parts per million of sodium chloride.

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* Cons. Engr., Atlanta, Ga.

The speaker was impressed by the comments of Mr. Holmquist regarding the decrease of typhoid rates. However, in commenting on water supplies of different municipalities, results should be compared with reference to the type of water treated. A comparison of typhoid rates between cities of the same size, in different parts of the United States, is not complete without all the information as to the type and character of water used.

H. Malcolm Pirnie,* Assoc. M. Am. Soc. C. E.—It has been said that coagulation basins which have a storage period of a few minutes, are adequate and preferable. It has also been said that the iso-electric point, as indicated by a P H value of 7, is the required hydrogen-ion concentration necessary for the best coagulation. These statements evidently come from experience with waters which coagulate rapidly and best at a hydrogen-ion concentration of P H = 7. Experience with other waters, however, has proven that long periods of coagulation and different P H values are sometimes required for good coagulation.

For many years, coagulation has been conducted with an acceptable degree of success, with the aid of standard methods of determining alkalinity and color by titration with methyl orange and comparison with color standards. Now, the new hydrogen-ion test has been developed. It is a new test and will give much interesting information, but it will not give at once a standard hydrogen-ion concentration to which all waters may be adjusted to obtain the best coagulation.

There are waters in which the nature of solids, particularly organic colloids, varies from hour to hour and from season to season, notably in waters which are polluted with industrial wastes and sewage. These waters do not contain a constant percentage of given organic matter, and the colloids may vary in their electrical properties and degree of adsorption for the added coagulant. In these circumstances, it is essential, with present understanding of chemical treatment, to have a balance wheel in the process, to equalize the effect of variations in the nature of these colloids, and large basins are useful in this connection. One particular plant, where there is a storage of 8 days for the coagulation period, is producing an effluent much better than it was possible to obtain in former years with a storage period for coagulation of 4 hours; in fact, it is now possible to take the water from the river with a color which is sometimes about 150, dose it, let it pass through this basin, and after passing the basin, give it a secondary dose of a small quantity of alum, and obtain a color of zero, after filtration. It is interesting to note that at the start, before any alum is added, this water has a P H value of less than 7. Before a floc can be obtained, it is necessary to reduce that P H value to between 5 and 6, which is considerably below the so-called iso-electric neutrality point of P H = 7. It is evident, therefore, that there is not a definite P H value for all waters, which may be used to determine the required alum dose. The required P H value will vary, depending on the kind and condition of colloids present in the water.

* Cons. Engr. (Hazen, Whipple and Fuller), New York City.

Colloid chemistry has suggested certain ideas which can be applied to water treatment. The physical action of colloids may be used to remove them without the aid of chemicals. This has been demonstrated recently in treating a well water containing a considerable quantity of iron in the ferrous state, which is converted into the ferric state by aeration, and simply passed upward through a definite bed depth of gravel. The ferric iron colloids which are so small that they will pass through filter paper and will not be removed in appreciable quantities by storage for many hours, are absorbed on the surfaces of the gravel stones. The bed is graded from 4-in. gravel at the bottom to ⅛-in. gravel on the surface, and the depth of the bed is 9 ft. An iron reduction from 7 parts per million down to a trace, with an almost complete removal of colloids, is effected. The adsorption of the precipitated iron by the wetted gravel surfaces and the greater adsorption of the iron colloids on the surfaces of the precipitated iron, are responsible for this almost complete removal of iron from the water. Physical forces play the major part in the purification.

It must be borne in mind that each new idea simply provides another tool with which to work, and that it is not a solution of the problems which engineers have in hand. The determination of the P H value may help to fix the nature of the colloids in a particular water, which would be a step toward working out for that water a definite method of treatment, but a method applicable to one water will not apply to all waters, or even to water at different locations in the same stream with intervening pollution from sewage or industrial wastes.

C. M. Baker,* Esq.—Mr. Emerson's discussion regarding the effect of gas-plant wastes on the taste and odor of chlorinated waters is interesting, especially since this condition has been a live question in the City of Milwaukee for some time. In the fall of 1918, before the speaker took up the work with the State Board of Health, exceedingly objectionable tastes and odors prevailed in the water supplied to the citizens of Milwaukee. In fact, it has been authentically stated that these conditions were so severe as to make the use of foods cooked in the water objectionable. The taste was ordinarily described as a medical taste, and did not resemble the taste of waters treated with an excess of chlorine. Furthermore, the tastes and odors occurred at infrequent intervals, not continuously, and the quantity of chlorine applied appeared to have little, if any, effect on the production of the objectionable conditions.

An extensive investigation was conducted, largely by the chemists of the Water Department and the City Health Department, which consisted in the collection of samples of water from the rivers and harbors in Milwaukee, and also of different industrial wastes. These samples were treated with an excess of chlorine, which later was driven off by boiling, and the test was made by tasting or smelling the samples thus treated. It was determined in this way that the taste-producing properties were confined exclusively to those wastes from industries producing coal-tar derivatives; other wastes seemed to have no effect. It was further determined that phenol was the most objectionable substance in producing these conditions. The investigators claimed that by

* State San. Engr., State Board of Health, Madison, Wis.

their method of testing, previously mentioned, the presence of phenol in a solution, as great as 1 part in 500 000 000, could be detected. At that time (1918), the State Board of Health and the Governor of Wisconsin were consulted. Numerous hearings were held, and, finally, by testing samples collected from Lake Michigan, the source of the trouble was traced directly to a Government phenol plant at Carrollville, about 15 miles from the Milwaukee intake. The matter was taken up with the Government and, in the fall of 1918, this plant was closed.

It was thought that the closing of the Carrollville plant had eliminated the objectionable conditions; but a slight recurrence of the odor was noticed in December, 1919, and again in March or April, 1920. This latter condition was more severe than that of December, and the State Board of Health was again called in consultation. At that time, the plant at Carrollville was operating on an entirely different basis, and was discharging no phenol wastes into the Lake. It was apparent, therefore, that the source of the pollution was elsewhere. The previous investigation, however, had failed to show the taste-producing quality in any of the samples collected in the harbor or the rivers. It was apparent, therefore, that the evidence was not conclusive against the two or three plants in Milwaukee known to produce wastes which were apt to cause the objectionable conditions. A complete investigation, therefore, was outlined and undertaken; thousands of samples were collected from the rivers and harbor, and tested.

Three rivers enter the harbor at Milwaukee, namely, the Milwaukee from the north, the Menomonie from the east, and the Kinnickinnic from the south. The Milwaukee Coke and Gas Company discharges its wastes into a slip tributary to the Kinnickinnic River and the Milwaukee Gas Light Company directly into the Menomonie River. The Milwaukee River and the Kinnickinnic River are flushed by tunnels from the lakes; the Menomonie River, however, is not flushed.

In the testing of samples, it was soon learned that there was only occasionally taste-producing qualities in the samples collected from the harbor outlet. By continuing the tests up the stream, the Milwaukee River was eliminated as a factor, but rather intense concentration was found in the Menomonie River—coming from the east—particularly just below the Milwaukee Gas Light Plant. On one or two occasions the taste-producing quality was observed in the water at the mouth of the Kinnickinnic River, and, on a few occasions, in the harbor outlet. A careful study of the relation of wind conditions with reference to the taste in the water supply indicated quite conclusively that the conditions necessary to cause tastes were a west wind, followed almost immediately by a southwest wind. Certain intensity and duration of the wind was also necessary. Furthermore, it was learned that samples collected from the harbor outlet showed taste on test only during westerly winds. It was concluded, therefore, that the conditions causing taste and odor in the Milwaukee water supply were: First, the accumulation of wastes from the Milwaukee Gas Light Company in the Menomonie River and from the Milwaukee Coke and Gas Company in the slip tributary to the Kinnickinnic River; and, second, by west winds carrying the accumu-

lated pollution out of the harbor, followed by southwestern winds carrying the pollution to the water intake.

The solution suggested was to provide for the continuous discharge of the wastes, thus prohibiting the intermittent discharges caused by changing winds. The wastes from the Milwaukee Gas Light plant, therefore, were pumped into a sewer discharging directly into the Milwaukee River, which sewer is flushed daily. The slip which receives the waste from the Milwaukee Coke and Gas Company was dammed and a pipe carried back of the dam for a distance of about 40 ft., to provide for the gradual discharge of the waste from this slip. Since these changes were made, there has been no recurrence of the objectionable conditions.

The situation in Milwaukee has been extremely serious. It has been so serious and such publicity has been given to the city's experience that much difficulty has been encountered in securing the installation of chlorine plants for the treatment of public water supplies in the State of Wisconsin. Some citizens of Milwaukee wished the chlorine discontinued although the necessity of this treatment was clearly demonstrated by an occurrence in 1916. One night, an employee of the Water Department shut off the chlorine without the knowledge of the Superintendent of Water-Works, or of the Health Department, and the apparatus was shut down for about 8 hours. There followed, during the next few days, between 50 000 and 60 000 cases of gastro-intestinal trouble, and within the next few weeks between 400 and 500 cases of typhoid fever, with between 40 and 50 deaths—very conclusive evidence of what chlorine is doing for the water supply of Milwaukee.

One more point regarding the effect which this condition may have over the United States. In the fall of 1919, the State Board of Health started a movement in Sheboygan to secure chlorination of its public water supply. The sewage from the city is discharged into Lake Michigan without treatment, and the water of the city is taken from the Lake untreated. The typhoid death rate for Sheboygan was about five times that of the State as a whole, and about three times that of Milwaukee. Serious opposition developed in the city, however, to the installation of chlorination, and it was only after the Department of Health issued a definite order and enforced it that such installation was secured.

George W. Simons,* Jr., Esq.—The discussion of Professor Winslow, treating of the reduction in the typhoid fever death rate, has been interesting to the speaker, but one or two points mentioned by him should be further amplified, especially by one familiar with health problems in the Southern States.

Health workers in the Southern States, and particularly those in the Southeastern States, are not so greatly concerned with the problems of water-borne typhoid infection as they are with infection from other sources. These States may be roughly classed as rural in comparison with the thickly settled Northeastern States the total area of which in some cases is far less than the area of many smaller counties of the Southern States. In a greater part of the

*Chf. San. Engr., State Board of Health, Jacksonville, Fla.

southern regions, municipal water supplies are taken principally from deep ground sources and, therefore, the chance of water-borne typhoid infection is considerably lessened. In all Southern States where soil-pollution activities have been conducted, the reduction of typhoid death rates has been clearly reflected in those areas. Therefore, at this time, the typhoid problem of the Southern States is largely one of rural sanitation. As these States become more thickly settled, however, and ground-water supplies become depleted or depreciated in quality and are replaced by surface waters, typhoid will become more of a water problem.

Professor Winslow has reproduced some interesting figures showing reductions in typhoid mortality rates in the registration area as they were originally, and, also, as they are now constituted. His remarks inferred that the recent admittance to the registration area of States having a lower sanitary standard was reflected by the lessening typhoid reduction in the present area, as compared with that of the original States. Unquestionably, the admittance of many Southern States into the registration area has had an effect on the rates, but, even so, one point deserves attention. Since 1910, fourteen States have been admitted to the registration area for deaths; of these, eight are Southern States, in which health activities are receiving more attention than ever before.

An examination of the typhoid mortality rates as issued by the Bureau of Census indicates that, during the period from 1900 to 1910, the typhoid death rate declined 13.3 points, and, from 1911 to 1919, 10.5 points. During the latter period, eight Southern States were admitted to the registration area.

Health work in the South is becoming more and more active, a fact which can be verified by an examination of the results accomplished. As stated previously, the typhoid problem in the South is largely one of rural sanitation, yet great strides are being made in the combat of this infection.

In Florida, for instance, the typhoid mortality rate was 27.8 in 1918; 18.4 in 1919; and 14.3 in 1920, which indicates a steady decline. The rate for 1921 will show an increase which has been a source of some apprehension to those studying it, but since the speaker has discussed the problem with workers from other States, the seriousness somewhat diminishes, because many States will experience a high typhoid rate this year; in fact, a representative of one of the largest Northern States has stated that he felt the typhoid mortality in his State would almost return to the 1913 figure.

There is still a final point which should be referred to, namely, the fact that much of the typhoid in a tourist State, such as Florida, is imported from elsewhere; in other words, Florida is charged with an undue typhoid mortality. During 1920, seven typhoid deaths were non-residents.

M. N. Baker,* Esq.—A few years ago, in discussing the diminishing typhoid rate, the speaker cast about for a figure which if attained might entitle a city or State to be placed on a typhoid honor roll. The figure decided on was 5 for a city and 7½ for a whole State. The rate has fallen so much since then that, in its summary of typhoid fever deaths in 1920, in cities of the United

* Assoc. Editor, *Engineering News-Record*, New York City.

States having more than 100 000 population, the *Journal* of the American Medical Association of March 26th, 1921, sets the typhoid honor roll mark at 2 deaths or less, per 100 000 population. With that low rate (taking into account a correction in a later issue) 11 cities appear on the roll, while 30 cities had typhoid death rates of 5 or less per 100 000. That is in striking contrast with the rate of 20 per 100 000 mentioned by Professor Whipple as that generally used some years ago in discussing the extent to which typhoid should be reduced in order to clear a city of the charge that the disease was due to the water supply. As the typhoid rate approaches 1 per 100 000 population, cities of less than that size have a zero rate now and then or, perhaps, for several years in succession. The latter has been true of Montclair, N. J., a city of about 30 000 population, where there has not been a death from typhoid for the past four or five years. Notwithstanding this absence of typhoid in Montclair, there has been much local agitation over the character of the water supply, although it is filtered and chlorinated. This shows what may be expected in many communities: Low typhoid or none at all, and yet more popular agitation over the water supply than when the typhoid rate was 20, 50, or even more.

With typhoid so largely reduced, there seems to be a chance of uncovering or exposing other areas of trouble—some of the more or less mysterious intestinal disturbances. Whether these are due to public water supplies is an interesting and important question which needs extensive and intensive investigation. The time seems to have passed when, in a progressive city which is utilizing the engineering and other technical knowledge now available, there need be great concern over typhoid, as far as water supplies go, provided only that the cities live up to the knowledge which is extant, both in the design and construction of water-works and in their operation and general practices as well.

Since this agitation in regard to water supplies in communities where typhoid has virtually vanished, comes very largely from the medical profession, or is backed by it, there seems to be occasion for urging on the medical men, if they are going to pursue that course and ask the communities to expend large sums of money for further water treatment, to make a careful study of their patients who have intestinal troubles, and try to find out some things which at present they do not know regarding these disturbances. In some communities, now and then, these intestinal troubles affect hundreds or thousands of people in a short time, but produce no typhoid. That was the case in Montclair in 1918 and, later, in Cambridge, Mass., which had a typhoid rate of 1.7 in 1919 and 2.7 in 1920. There is still plenty for engineers to study, but members of the medical profession should go further in their studies than they have yet done, if they are to ask engineers and the public to increase the already large expenditures to improve the quality of water supplies.

H. N. Bundesen,* M. D.—There is undoubtedly a great deal in the statement of Professor Winslow that the typhoid rate in the United States is near that of European countries. The rates in this country are probably far closer to the European standards than Professor Winslow's figures would show. In

* Chf., Bureau of Water Safety and Typhoid Control, Dept. of Health, Chicago, Ill.

Europe, where post-mortems are resorted to so frequently, the causes of death are given more accurately and if the same scrutiny pertained in the United States, many communities would have rates of 1 per 100 000.

During the past ten years, tabulation in Chicago, Ill., shows that the average duration of typhoid is 21 days before death. In Chicago, when a typhoid death certificate is presented and the case had not been reported, and if the certificate shows that the person has only been sick two or three days, it is looked on with suspicion and is investigated. Of the ten death certificates held up by the Department during the past several years, the bodies were posted by the coroner, and the cause of death in each instance was abortion or similarly concealed conditions. It would be interesting if Professor Winslow could take the 1916 and 1919 figures which he has presented and ascertain how many of those deaths occurred among patients who had only been sick three or four days.

A question of vital importance to the health authorities of Chicago at the present time, has been presented relative to the use of chlorine in connection with the water supply. More than 500 000 lb. of chlorine are used in a year, and every day more than 800 000 000 gal. of water are pumped. At times, complaints are received regarding the taste of chlorine. The water supplied from Lake Michigan to a community surrounded as Chicago is, must be protected. The water is not filtered and, in consequence, chlorine is used to protect the people. The health authorities have not been able, at any time, to trace any injurious effects to the use of chlorine. It is true that at times people will be told by their family physician that the trouble is due to chlorine, but the speaker is of the opinion that they are simply told this for want of a better diagnosis.

Another feature which might be of importance is the presence of *B. Welchii,* which has been found in the drinking water before chlorination, and at times has been taken from the taps after chlorination. Of course, this bacillus is a spore bearer and is not killed with chlorine.

Charles Haydock,* Assoc. M. Am. Soc. C. E. (by letter).—In the past, the subject of industrial water supply has not received the attention to which it is rightly entitled. During recent years there has been a decided change of opinion coincident with the growing realization of the dependence of many industries on water supply, and the effect that the quality of the water has on the operating costs of such industries. Until comparatively recently only a few industries gave serious consideration to the quality of their water supply, or to the man who, in many instances, realized most forcibly the effect of that quality, namely, the operating engineer whose coal bill was unduly large, whose boiler was pitted unduly, or whose feed-water heaters and hot-water systems were destroyed within a comparatively short time.

Although it is true that the protection of life is the most important function of a water purification plant and, in general, is the fundamental reason for its construction, the chemical effects of the proposed system of purification should be carefully investigated before a decision is made, in order that

* Engr., Water Companies, Philadelphia, Pa.

an unnecessary burden may not be placed on the various industries that will use the purified water. The importance of this feature and the frequency with which it has been disregarded in the older plants can be realized by considering the number of such plants producing an effluent more actively corrosive than the raw water from which it was manufactured. It is also to be noted that the desirable qualities of a water for industrial purposes are not always the same as those for domestic purposes and that the quantities required for the former uses are far greater than for the latter, even in a large city. The use of alum, for instance, increases the hard scale-forming qualities of a water when used for boiler purposes.

The methods of reducing the damage caused by industrial water supplies of poor quality should not be made more difficult by ill-advised treatment designed to render the water safe for domestic uses. Professor Whipple's statement that the elimination of the corrosive qualities of water and the purification of water supplies for industrial purposes is a refinement, is unwarranted, and, fortunately, is to-day held by a decreasing number of individuals. Certainly no one connected with the operation of any industry that uses a material quantity of water will agree with such an opinion.

The classification of waters as "active" and "quiet", or more commonly called "passive", as mentioned by Mr. Hazen, is indeed helpful. In too many cases active waters have been accepted without question, as comparatively few users know that passive waters are obtainable, and, therefore, public sentiment is lacking to compel rendering the waters passive. Active waters constitute a serious handicap to any community required to use them. The corrosive effects in the destruction of service lines, house plumbing, and hot-water piping, are important items of expense in the annual repair bills; after-coagulation is also induced by certain active waters. These effects reduce the capacity of pipe lines to a serious extent, frequently causing the entire stoppage of service lines. In general, the writer believes that it can safely be said that those who are in responsible charge of progressive industrial plants seem to realize the value of soft passive waters to a far greater extent than the designers of many domestic water-treating systems. This has been reflected in the demand for water-treating plants for industries, which, in many instances, use waters already purified for domestic purposes as their raw water supply.

Probably the most dangerous active waters are in the Pittsburgh District.* The value of soft quiet water seems to be more clearly recognized there than anywhere else. Few unpolluted streams are available, practically all the larger watercourses carrying large volumes of coal-mine drainage, which render them acid continuously or except during the periods of the high spring run-offs. All the major water-using industries in this District find it necessary to maintain water-treating plants which are expensive to operate, but, in some cases, are able to render the waters usable for certain purposes. In the writer's judgment, it can safely be said that from an industrial viewpoint, no treated water is a satisfactory substitute for a naturally pure water supply.

The larger creeks in the Pittsburgh District, in general, are so seriously polluted that no attempt is made to use them. In one instance, a certain

* Recently discussed in the *Journal* of the American Water Works Association.

creek is used for condensing purposes. The tubes of the condensers are of acid-resisting bronze and the body is lined with lead and cement. In some instances, in spite of such precautions, these condensers have been destroyed within a period of three months.

The Youghiogheny River flows into the Monongahela a few miles above Pittsburgh and has a drainage area of 1 732 sq. miles. It is acid practically every day in the year, and has reached a maximum of 22 grains per gal. The Monongahela River, the drainage area of which is 7 340 sq. miles, is very hard, but, in general, is alkaline. Its hardness is greatly increased during periods of low flow and, at such times, it frequently becomes acid. This also occurs following a rise in the river, due to flushing the upper pools. As a safeguard many of the steel mills which use this water for cooling their rolls, add lime, although, in one instance, the quantity used is approximately 100 000 000 gal. per day.

The supply of the City of Pittsburgh, which is taken from the Allegheny River, is of much better quality, but as long ago as 1912, the Managing Engineer of the Municipal Water-Works called attention to the progressive lowering in the alkalinity of this river and the eventual necessity for taking measures to meet this situation. Since that time there have been no very dry seasons. During the first year in which a protractive deficiency of rainfall occurs, particularly if it should be accompanied by high temperatures, no doubt serious difficulties will be experienced in the operation of the Pittsburgh filter plant.

Although these acid waters are highly active, they are practically never used for industrial purposes without treatment, as they would rapidly destroy boilers and all other metallic structures with which they came in contact. It is interesting to note that, in general, waters which are active, but acid free, do not destroy boilers, as the dissolved gases are eliminated rapidly. If a feed-water heater is used, they are almost certain to be eliminated at that point with a consequent destruction of the heater. As an example, it can be stated that in the office building in Philadelphia in which the writer is located, the hot-water piping was destroyed within a comparatively short time and new piping had to be installed, together with a means of removing dissolved gases, which it is hoped will eliminate the corrosion.

The City of Altoona, in Central Pennsylvania, uses a water supply that is acid at all times. This water is practically impossible of use for industrial purposes without extensive treatment, and it is exceedingly expensive to maintain the house-plumbing systems. Meters have recently been installed on all taps, and it will be interesting to note the effect of these acid waters on the meters. It has been proposed to attempt to correct this difficulty by carrying the drainage from the coal mines, which contaminates the supply, below the reservoirs by a sewer. As a result of installing meters, the city supply now has a sufficient margin over the demand to permit this to be done, and according to recent newspaper accounts an effort will be made to compel the mine owners to pay for the cost of this work, which was rendered necessary by their mining operations.

Legal actions to protect the purity of a water supply from contamination by coal-mine drainage are also in process in Fayette and Westmoreland Counties in Pennsylvania. A water company constructed a storage reservoir and an extensive system of piping. Many years later, coal mines were opened above the storage dam. The present litigation has been instituted by the water company and allied interests in an effort to prevent the destruction of its valuable water supply and other properties. This case has been described as the most important pending litigation in Pennsylvania, and its importance is further emphasized by the fact that the Attorney General, acting on behalf of the Commonwealth, has filed petitions for leave to intervene. Argument has been held on these petitions, but so far no decision has been rendered.

The quiet waters between the Alleghanies and Rockies are very hard, and to those accustomed to the softer Eastern waters, almost unbelievably hard. The engineer of water service of an important Western railroad has rather facetiously remarked that some of these waters are so hard that it is customary to analyze them for percentage of moisture rather than for solids.

It is most unfortunate that there is no well established method for treating hard waters satisfactorily. The commoner systems in use simply alter the solids in solution, usually increasing them. Barium is the most common chemical which produces a double precipitation and consequent elimination of the electrolyte, but this treatment is exceedingly costly and the soluble barium salts are poisonous under certain conditions, so that in general this method is not available. If the hardness is high, use of the water for many industrial purposes is prohibitive without some form of softening. This process does not remove difficulties any more than it reduces total solids in solution and, in addition, the use of treated water introduces many problems not encountered in using naturally pure supplies. If the activity of a water is due to acidity, it can be neutralized by the addition of proper chemicals and greatly improved in quality for industrial purposes. This benefit is secured at the expense of an increase in hardness which, at times, may be so great as to be prohibitive, as is the case with so many streams draining the extensively developed coal-mining regions.

The legal aspects of stream pollution are of the greatest importance and are now becoming more widely realized. The Legislative Act creating the Pennsylvania Department of Health, of which C. A. Emerson, Jr., M. Am. Soc. C. E., is Chief Engineer, states in part: "This Act shall not apply to waters pumped or flowing from coal mines or tanneries", thereby eliminating the two industries which have caused the most serious pollution of streams in Pennsylvania. The presence of vast areas of high-quality coal is beyond question one of the foundations of the industrial supremacy of Pennsylvania, and it is unthinkable that legislation should be enacted, which would seriously handicap this vital industry. On the other hand, there are so few unpolluted streams left in the developed coal regions of Pennsylvania that something must be done unless the future industrial development of the State is to be hampered. This condition has become so serious at times that railroads have been greatly delayed in hauling the coal produced by mines which polluted the streams,

the water of which is used for boiler purposes, to such an extent that all the available locomotives were out of service for repairs.

One of the strongest laws on the statutes of Pennsylvania for the prevention of stream pollution by industries is designed to protect fish, and is administered by the Commissioner of Fisheries. In 1915, in addressing the Pennsylvania Water Works Association, the Commissioner called attention to the difficulties of enforcing this Act by reason of the low penalties provided and the unwillingness of Courts to impose these penalties even in the most flagrant cases where conditions could be remedied with comparatively little difficulty or expense. During the last session of the Legislature, an unsuccessful effort was made to amend this Act and raise the penalties so as to permit enforcement of the law. As a result, industrial pollution will continue unabated until at least the next session of the Legislature. Practically all forms of industrial pollution of streams destroy the fish life therein, and the effect of this is to render difficult the enforcement of any of the other laws designed to protect the food and game fishes. This condition was particularly apparent during the low water of the summer of 1921, along the Susquehanna River in the vicinity of Williamsport, when large numbers of fish were killed. As a result, the average individual was unable to see why he should be punished for taking a few game fish illegally when industries destroyed many thousand times more fish without incurring any liability. Any law that is not enforced is a menace, in that it leads to disregarding the law in other cases where the effects of such disregard may be even more serious. Another difficulty encountered by the Commissioner in his efforts to prevent pollution is the lack of a suitable means to treat water contaminated by various industries, chiefly coal mining. It was stated that the managers of many industries were willing to install any system of purification satisfactory to the Commissioner, but as no method was then known or has yet been developed, nothing was accomplished, and he did not feel warranted in closing down the industries until a suitable method was developed.

Referring again to the Pittsburgh District, the Youghiogheny River which is uniformly acid, has been devoid of fish life for many years. The Monongahela is said still to contain a few hardy fish which probably live in the vicinity of the mouths of the purer tributaries. The Allegheny is able to support fish life, as is also the Ohio some miles below Pittsburgh, due to the mixing of the Monongahela and Allegheny. A rise in the Monongahela River, unaccompanied by a corresponding rise in the Allegheny, will flush accumulated acid from the pools of the Monongahela and carry it far down the Ohio to points where the waters are ordinarily acid free. The result of this is that large numbers of fish are killed, and the occurrence of this phenomenon has been reported as far as Cincinnati, Ohio.

Several bills designed to prevent the discharge of oil and oil wastes into harbors and navigable streams were considered by the special session of Congress in 1921. Hearings have been continued and, undoubtedly, similar measures will be introduced later as some legislation is certainly needed to prevent this serious menace to harbors and beaches. These bills are being opposed by

coal operators and manufacturers' associations on the grounds that they might be considered applicable to industrial wastes other than oil sludge. At a hearing, held in Washington on December 7th, 1921, Secretary Hoover testified at length on the seriousness of the situation, recommending that at present legislation be passed applicable only to the oil wastes. Conditions with respect to other industrial wastes, he stated, were so different that they should be thoroughly investigated before any action is taken. The difficulties imposed on communities and industries compelled to use the waters polluted by industries were not brought before the Committee.

Several years ago, the U. S. Engineer's Office at Pittsburgh made an extensive investigation of the acidity of the local rivers, and as a result of that investigation a bill was introduced into Congress to prevent the discharge of acid mine wastes into navigable rivers or their tributaries. Such drastic legislation, naturally, did not pass, but it is to be hoped that, in the near future, some provision will be made for the abatement of what was once a nuisance, but what has now become a menace.

The availability of a satisfactory water supply is an important feature in determining the location of large industrial plants, as is indicated by inquiries received at the offices of industrial agents of large railroads. The report of the recently completed Super-Power Survey recommends that generating stations be built adjacent to the coal mines wherever condensing water is available.

In view of the importance of pure water for industrial purposes, and the lack of satisfactory methods of purification, the writer believes that there is need for research to be conducted on a broad scale and pursued to a conclusion, preferably by some such Federal agency as the Bureau of Standards. An investigation of this nature was conducted by the Pennsylvania Department of Health. As no results were announced, it can be assumed that the investigation was unsuccessful. It was indeed most unfortunate that such an important investigation which was being conducted at such a small cost, should be discontinued on the grounds of economy, as such results are a powerful deterrent to any one contemplating work of this nature.

The greater fall in rural typhoid in the original registration area, as mentioned by Professor Winslow, may have been due to cases carried by media other than water, or to the general rise in the standard of living during recent years.

It has frequently been noticed that typhoid fever in communities served by private water companies is at times charged to the water supply without investigations being made. This has occurred in several instances with which the writer is familiar although analyses showed the public water supply to be satisfactory while analyses of wells and springs used for drinking purposes by many consumers of the water company showed the presence of *B. coli* in very small samples. Fair and impartial judgment would seem to require that all other possible sources of contamination should be investigated, such as milk, ice, ice cream, soda water, raw foods, etc., before the outbreak is charged to the water company.

It is unfortunate that the local physicians do not always aid in quieting the hysteria which so often develops following an outbreak of intestinal disease. Valuable service could be rendered to the communities by calling attention to other sources of possible contamination in cases where the water analyses are satisfactory. In some instances, this, no doubt, would lead to an earlier checking of the spread of the disease.

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Paper No. 1489

A REVIEW OF IMPORTANT DEVELOPMENTS IN THE SCIENCE OF CADASTRAL RESURVEYS AS EXECUTED BY THE UNITED STATES GOVERNMENT, WITH ETHICAL DISCUSSION THEREOF

By Howard Richards Farnsworth,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. M. Douglas, D. E. Hughes, W. J. Lightfoot, Clay Tallman, S. V. Proudfit, H. B. Patten, J. F. Le Baron, and Howard Richards Farnsworth.

Synopsis

Owing to the great number of interesting phases appropriate for discussion in any general review of the science of cadastral resurveys, this work has necessarily been restricted, and is confined to a study of some of the more recent and important of the technical improvements in field and office methods, as developed in the Cadastral Engineering Service of the United States General Land Office.

It must be assumed that the reader is familiar with the long established official method of execution of original public land surveys, which produces a rectangular network of 36 sections per township 6 miles square, with monuments established at intervals of ½ mile. At present, the original areas are nearly all subdivided and platted, and the greater part of what was once a vast undeveloped public domain has passed into private ownership. The rapid development of this area and of its latent wealth depends largely on a quiet title and undisturbed possession which, if not secure, immediately retards the growth of the mineral, lumber, agricultural, and live-stock interests that in large measure form the basis for credit at the banks and that National wealth which is the bulwark of the country.

Until 1910, the Government executed these original surveys by entering into contract with a successful bidder, generally on a mileage basis. An unfortunate result was the frequent careless and sometimes fraudulent work, espe-

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cially in the early surveys, the natural desire of the deputy surveyor being to execute his contract with the minimum of expense and danger to himself. The early surveys were often a mass of errors, sometimes with no monuments set at all, a fictitious plat and field record having been prepared and filed, which described the true extent and character of the lands represented with only remote accuracy. Again, especially on the Western plains, Nature and the elements have gradually obliterated the corner monuments, important boundaries and whole counties being thus affected. Finally, when settlement began, a vain search for corners commenced, strife ensued, and unending lawsuits developed among the claimants and owners of the lands.

Other great surveying departments of the Government, such as the United States Coast and Geodetic Survey, have always used the most refined methods, a direct method of execution instead of contract, and have taken ample time to measure and calculate distances and angles properly. Yet, when errors are found in their work, no law intervenes to prevent their correction and the moving of the monuments. Corners of public land surveys, however, on which private rights have been based, cannot be changed. Other surveys remain as before, subject only to their scientific authors, whereas the cadastral survey becomes a creature of the Courts.

In order to satisfy the ever-increasing demand for the scientific and legal treatment of this "error", Congress has provided for the Federal execution of cadastral resurveys. For the past eleven years, this complicated and exacting work has been successfully prosecuted by the cadastral engineers of the Government, who are applying only the most approved scientific and legal methods. It is believed that every civil engineer having to do with field work, will find the discussion of the subject, as set forth in this paper, to be of interest and value.

ADMINISTRATION

Organization and Duties

The tenth anniversary of the inauguration of the direct system of surveys and resurveys by the General Land Office was reached on June 25th, 1920, on which date, in 1910, Congress discontinued the old contract system* and the work was placed under the direct supervision of the Commissioner of the General Land Office. The record of all classes of accepted surveys under the new system, for the past ten years, is estimated at 100 109 570 acres, or an average of 10 000 000 acres per annum, the high mark having been reached in 1915. The aggregate area resurveyed during the fiscal year ending June 30th, 1920, was 2 514 306 acres.†

The administration and technical direction of this stupendous work is vested in a small group of civil engineers in what is known as the Division of Surveys and Resurveys. With the advice and counsel of a Board of Law Review, these engineers act for the Commissioner in the attainment of correct

* For an exposition of this system, see the paper by C. L. DuBois, Mass. Inst. of Technology *Quarterly*, Vol. XII, No. 4 (December, 1899).

† Report of the Commissioner of the General Land Office to the Secretary of the Interior, June 30th, 1920, pp. 19 and 127.

execution and practical results in what is undertaken. This Division is the administrative unit which undertakes the administration, supervision, technical control, and execution of the survey of public lands together with other surveys involving the boundaries of Mexican and Spanish private land grants, of State and international boundaries, and resurveys authorized by law.*

Fig. 1 is a diagram of the administrative organization of the Cadastral Engineering Service of the General Land Office as of 1919, showing the relative supervisory responsibilities.

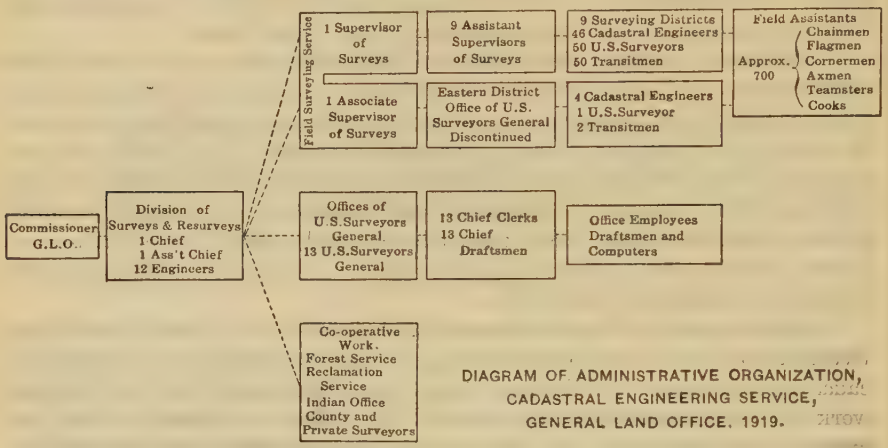


FIG. 1.

AUTHORITY OF LAW

In general, the legislation provided by Congress for the execution of cadastral resurveys under this organization may be divided along the lines of the application thereof to two general conditions, first, where title to more than 50% of the land to be resurveyed remains in the United States, in which case the Government bears the entire cost; and, second, where disposals are in excess of 50% of the total area, in which case the private holder bears his share of the expense as well as the Government, proportioned in accordance with the acreage held. This service which has become invaluable to the Western States, has recently been made available to the Central and Eastern States where unending boundary disputes are still found to obtain, although for many years the lands have been held in private ownership.

The first general Act was approved by Congress on March 3d, 1909, and was entitled "An Act Authorizing the Necessary Resurvey of Public Lands." It reads in part as follows:

"That the Secretary of the Interior may * * * cause to be made * * * such resurveys or retracements of the surveys of public lands as * * * he may deem essential to properly mark the boundaries of the public lands remaining undisposed of: Provided, that no such resurvey or retracement shall be so executed as to impair the *bona-fide* rights or claims of any claimant, entry-

* Further information on this subject may be had by reference to the decision of the U. S. Supreme Court in United States *vs.* Morrison (240 U. S., 210–212, inclusive); to the 1919–21 files (Form No. 28), of the U. S. Bureau of Efficiency; and to the report dated March 12th, 1920, of the Federal Reclassification Commission.

man, or owner of lands affected by such resurvey or retracement." (35 Stat., 845).

The second general Act, entitled "An Act Authorizing the Resurvey or Retracement of Lands Heretofore Returned as Surveyed Public Lands of the United States under Certain Conditions" (40 Stat., 965), was approved September 21st, 1918. Work under the latter Act is undertaken on the application of the owners of three-fourths of the privately owned lands, or of any Court of competent jurisdiction.

Restrictions Imposed

From a review of Congressional legislation, it is evident that:

Where private rights have attached to boundaries of the public lands, as duly accepted by the Commissioner of the General Land Office, these boundaries are unchangeable.

Original monuments of all classes on such boundaries must stand as the true corners which they were intended to represent, in so far as such rights are affected, irrespective of whether their actual positions agree with the record of the original field notes.

Where corners on such boundaries have become obliterated or are lost, they must, in a resurvey, be restored in their original positions, according to the best available evidence thereof.

The jurisdiction of final review in all matters of private ownership affected by a resurvey, is vested in the Courts. Thus, it is evident that, in suits involving boundary disputes, the rules of procedure laid down to guide the U. S. Cadastral Engineer in his execution of resurveys must be in harmony with the leading Court decisions, in order that, when properly applied, the Courts may accept without question the boundaries thus determined in so far as they represent the true location of a particular tract of land intended to be conveyed by a patent.

IMPROVEMENTS IN THE FIELD

In Equipment

The modern practices developed and utilized in the execution of Federal cadastral surveys are in keeping with the orderly advance in refinement and precision to be found in the methods used in all other engineering and scientific activities, when compared with the practice of years ago. A 1919 edition of the "Manual of Surveying Instructions", which embraces six of the ten chapters contemplated for the completed edition, completely revises all previous issues with additional new material. This edition has been prepared by a Board, created by the Commissioner, composed of four members of the Engineering Staff of the General Land Office, together with a field officer, which, as a body, deserves much credit for expanding and establishing the practice of a highly specialized science.

A few of the more convenient and useful forms of observation as taken from these recent "Instructions",* will be given in the pages that follow. Some of these forms are not in general use and will appeal to the field engineer.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", General Land Office, 1919, p. 23.

Alignment

Improved Solar Transit.—The engineer's transit, as equipped with the Smith solar attachment, has been developed in the General Land Office to a state of efficiency which fully warrants the adoption of this model as a standard instrument. In improving the construction, special attention has been given to provide suitable means by which working parts may be properly adjusted to insure uniform accuracy in the results attained. No detail has been omitted, and the perfected arrangement of the working parts permits a precise and rapid adjustment in the field, in a simple manner readily understood by any competent operator and entirely free from any uncompensated or residual errors. Adequate provision has been made for a maximum protection of the delicate working parts, with due attention to compactness and to a proper distribution of weight.

A result of special tests in this service, in which large numbers of every form of solar model are constantly in use, has proven the Smith solar attachment to be far superior in efficiency to all other forms. In improving the construction of this solar attachment to obtain the standard instrument, the process of evolution from the old to the new model has been gradual and tedious, each change being put to the test of months of actual field use.

In the improved construction, a regular light mountain model, full engineer's transit is used, on the east standard of which is mounted the solar attachment. The essential features of the important improvements over older models are as follows:*

1.—The solar has been mounted on an instrument having V-shaped standards, thereby adding much to the stability of the attachment.

2.—The base-plate of the solar is mounted on three foot posts, thus relieving the strain due to imperfect adjustment of the older models having a four-point base.

3.—The position of the base-plate is adjustable by opposing capstan nuts on the foot posts, each with a countersunk ball washer, thereby obtaining positive adjustment altogether free from strain on the capstan nuts.

4.—The three-point base forms a right-angle triangle, with one side horizontal and one side vertical, thereby permitting adjustment in either of two directions: (*a*), One about a horizontal axis, and (*b*), one about a vertical axis, either without disturbing the other.

5.—The axis of the latitude arc is arranged so that its position may be tested with a striding level without removing the auxiliary telescope.

6.—Both the latitude arc and auxiliary telescope are hung beneath the latitude axis, thereby lowering the center of gravity of the attachment and giving much greater protection to the delicate working parts.

7.—Suitable capstan nuts have been placed at one end of the auxiliary telescope to provide for its proper adjustment, with respect to the axis of the latitude arc.

* A. D. Kidder, "A Description of the Smith Solar Attachment, as Recently Improved for the Surveying Service of the General Land Office", published under the direction of the Commissioner, General Land Office, 1915, p. 5.

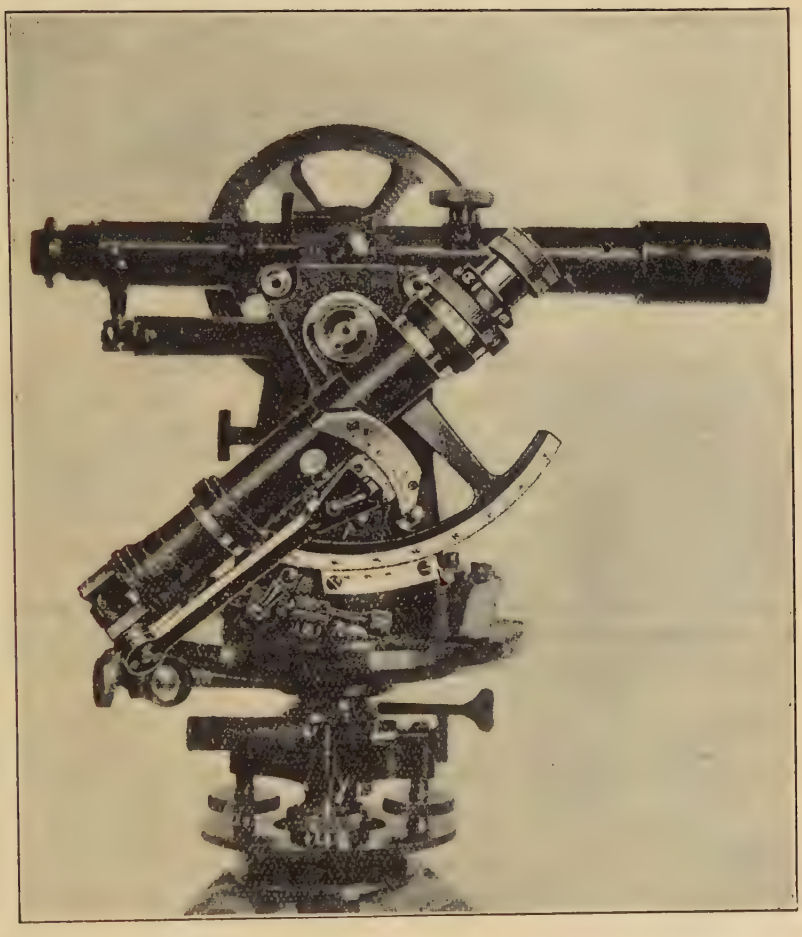


Fig. 2.—The Improved Solar Transit.

8.—Improved interlocking devices have been placed on the latitude and declination arcs, verniers, clamps, and tangent motions.

9.—The mirror may be swung around instantly to permit direct sighting through the auxiliary telescope.

10.—Absolute freedom of motion of the various working parts each to perform its own function, and each one independently, quickly, and permanently adjustable.

Maximum efficiency may be expected and obtained from the operation of the solar attachment only when the attachment itself, as well as the transit on which it is mounted, is in proper adjustment and when so adjusted efficient meridional performance may be obtained. The recent improvements have added greatly to the precision and ease with which the adjustments may be accomplished. These adjustments are described in detail in the advance sheets of the "Instructions" issued under the direction of the Commissioner, previously referred to, and are well worthy of adoption by all engineers interested in time, latitude, and azimuth determinations by the use of this model. (Fig. 2.)

Measurement

Triangulation and Stadia.—The Federal engineer is authorized to obtain distances across water and over precipitous slopes by the use of appropriate triangulations or a properly safeguarded stadia method. In the former, care is exercised in the selection of the measured base and the adoption of the best possible geometric proportions of the sides and angles of the triangle. When it is desired to determine the value of any angle with a precision less than the least reading of the instrument, the method of repetitions is utilized. In the use of the stadia method, the wire interval or ratio is required to be determined in the field by frequent tests and under working conditions, in comparison with steel tape measurement, solving the following formula* for the value of the wire ratio when the horizontal distance is known:

$$\text{Horizontal distance} = K\ r \cos^2 V + (c + f) \cos V$$

in which

"horizontal distance" = the distance from the center of the instrument to the rod;

K = the wire ratio;

r = vertical rod reading;

c = distance from center of instrument to object glass;

f = distance from plane of cross-wires to object glass; and

V = observed vertical angle.

Long Steel Tape and Clinometer.—Where it is possible, the most approved method of measurement, however, is that involving the use of steel ribbon tapes from 2 to 8 chains in length, checked frequently against a suitable reference standard.

* "Tables and Formulas for the Use of U. S. Surveyors and Engineers on Public Land Surveys," Commissioner of the General Land Office, Second Edition, 1913, p. 221.

The long tape when used is properly aligned and stretched so that the measurement may be made on the slope. One of the most rapid and reliable methods of measurement is thus obtained by the skillful use of the long steel tape. The transit is used to determine vertical angles of particularly sharp slopes, the angle of lesser slopes being determined with the clinometer. Slope distances are then reduced to true horizontal distances, and the entire operation is recorded. Rapidity in the reduction of slope measurements is obtained by the use of appropriate reduction diagrams.

In Methods

Beginning with 1890, the inhibition against the use of the primitive magnetic needle for the determination of direction increased, and, in 1894, it was made absolute for all classes of lines on public land surveys. In the execution of the present-day survey, all bearings are determined with reference to the true meridian as defined by the axis of the earth's rotation. Such meridian determinations are required at the beginning, at necessary intervals, and at the conclusion of each work. The instruments are of the latest design and are used only on the approval of a supervising officer after they have undergone a satisfactory test in the field. It is now truly the purpose of the Government to accomplish final results in all its lines of survey, especially in those affecting boundaries of private vested rights.

Different forms of time, latitude, and azimuth observations have been developed and re-arranged to facilitate work under all conditions encountered in the field, most of which are referred either to the sun or to Polaris. Time is readily determined with an error not to exceed 10 sec., while latitude and azimuth are readily available with an error not to exceed 1′ 00″, small errors in assumed longitude being neglected in such determinations.

Some of the more useful of the advances in methods developed and authorized for the execution of official cadastral surveys, as taken from the general instructions issued by the Commissioner of the General Land Office in 1919,* are given in the pages which follow.

Declination.—The advantages of a graphic method for ascertaining changing declinations of the sun, corrected for refraction in polar distance, are to be found in the practical elimination of errors of computation, ease of checking, and in the fact that actual values are obtained without interpolation. This method is illustrated by Figs. 3 and 4. Fig. 3 is a diagram of the sun's declinations for March 20th, 1912; Lat. 37° 30′ N.; Long. 7 hours 30 min. W.:

Declination, Greenwich noon = 4:30 A. M.	= 0° 11′ 14″ S.
Difference, 10 hours = + 593″	= 09′ 53″ N.
Declination at 2:30 P. M.	= 0° 01′ 21″ S.

and Fig. 4 is a diagram of the sun's declinations for September 23d, 1913; Lat. 47° 30′ N.; Long. 6 hours 18 min. W.:

Declination, Greenwich noon = 5:42 A. M.	= 0° 03′ 55″ N.
Difference, 10 hours = — 585″	= 9′ 45″ S.
Declination at 3:42 P. M.	= 0° 05′ 50″ S.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of Public Lands of the United States", 1919, pp. 23, 48–49.

In Figs. 3 and 4, the horizontal lines represent each hour of the day, and the vertical lines represent 0° 01′ intervals in declination, the latter being numbered to suit the range of the sun's declination for the date. Two points

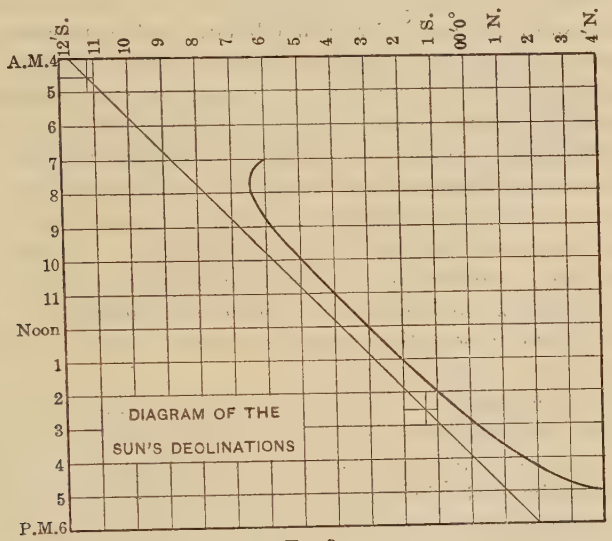


FIG. 3.

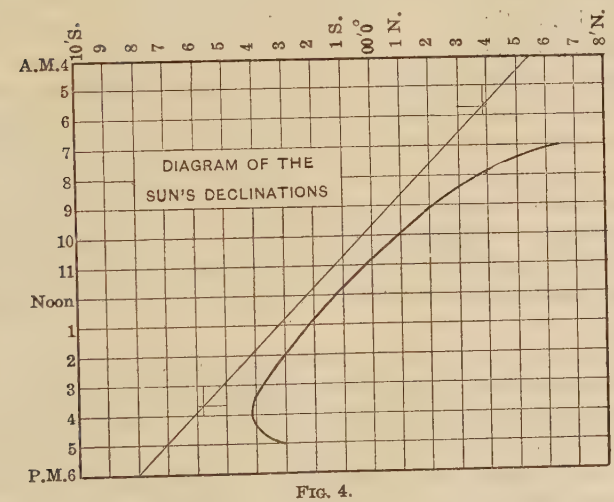


FIG. 4.

are marked on the diagrams to agree with the true declination of the sun. The first point is marked with the argument of declination agreeing with that given in the Ephemeris* for Greenwich apparent noon, that of time agreeing

* "Ephemeris of the Sun and Polaris, and Tables of Azimuths and Altitudes of Polaris", Commr. of the General Land Office, pub. annually since 1910.

with the local apparent time corresponding to Greenwich noon. The second point is marked to agree with the proper declination and time, 10 hours later. A straight line connecting the two points will then form the locus of all points which indicate the sun's true declinations for the date, for local apparent time. The proper refractions in polar distance are then scaled from the straight line toward the north for each tabulated value, morning and afternoon, as given in any correct table of values of mean refractions in polar distance, appropriate for the latitude of observation and declination of the sun. The locus of these points as plotted is a smooth curve representing the sun's declinations corrected for refraction in polar distance. The scale used for plotting the refractions must equal the scale of the intervals of 0° 01′ in declination, and the refractions must be laid off parallel to the horizontal lines and not normal to the line of true declination. A reading from the curve at the point corresponding to the time of observation will then give the proper declination for use with the solar attachment. For use in the reduction of altitude observations, the true values of the sun's declination will be taken from the straight line.

Time.—It is necessary for the cadastral engineer to know the exact apparent time of all his observations taken on the sun and the true local mean time of all observations taken on Polaris. When comparison with standard telegraph time is impracticable, direct determinations become necessary, and the method used will be either an altitude observation of the sun for apparent time, meridian observation of the sun for apparent noon, meridian observation of a star of appropriate magnitude, and declination for local sidereal time, or time secured directly by use of the solar attachment. Determinations by the latter method, however, are restricted to use in observations on Polaris at either elongation, and, of course, are not sufficiently accurate for observations by the hour-angle method. The meridian observation of the sun is by far the most convenient reliable method of time observation. The observing program for this method is as follows: Determine the meridian by the best means at hand and compute the altitude setting for the sun; level the transit, place the instrument in the meridian, and elevate the telescope to the altitude of the sun's center; note the watch time of the sun's west limb tangent to the vertical wire; and note the watch time of the sun's east limb tangent to the vertical wire.

Take the mean of the readings for the watch time of apparent noon from which to compute the watch error, local mean time. If the observation fails for either limb, the reduction to the sun's center is accomplished by adding or subtracting 68 sec. A refinement in this amount is had by reference to the Ephemeris* for the time of the sun's semi-diameter passing the meridian for the date of observation. The setting for the approximate altitude of the sun's center is:†

$$V \neq 90° - \phi \pm \delta$$

* "Ephemeris of the Sun and Polaris, and Tables of Azimuths and Altitudes of Polaris", Commr. of General Land Office, pub. annually since 1910.

† "Advance Sheets of a Revision of the Manual of Instructions for the Survey of Public Lands of the United States", 1919, p. 65.

The field record appears somewhat as follows:* August 14th, 1909, in latitude 37° 16′ N.; longitude 102° 16′ W.:

Setting:	90° 00′
$\phi \neq$ (—)	37° 16′ N.
$\delta \neq$ (+)	14° 25′ N.
$V \neq$	67° 09′

	h.	m.	s.
°+ Watch time of transit, W. limb =	11	59	22
+° Watch time of transit, E. limb =	12	01	32
Watch time apparent noon =	12	00	27
Apparent noon =	12	00	00
Equation of time =	+	4	33
Local mean time of apparent noon =	12	04	33
Watch slow of local mean time =	0	4	06

In this and in other observations hereinafter given, the following analytical notation is used, to follow that used in the Commissioner's "Instructions":

Let $\neq$ = an inequality which approaches equality;

V = observed vertical angle; in altitude observations on the sun, the mean observed vertical angle to the sun's center.

h = true vertical angle to the sun's center, or to Polaris, in altitude observations, after correction for refraction: $h = V$ — refraction in zenith distance. In altitude observations on the sun, a refinement is had by adding the value of the sun's parallax = 8.9″ cos V, opposite in effect to refraction.

ζ = true zenith distance of the sun's center = 90° — h.

ϕ = latitude of the station of observation.

λ = longitude of the station.

δ = declination of the sun or Polaris.

Latitude.—A definite knowledge of the true latitude is very important in the use of solar instruments. No lack of reasonable precision is allowed in an accepted latitude and, for this reason, a considerable series of latitude determinations are taken on every large work, which, on comparison, will produce a satisfactory mean. Altitude observations of Polaris at the upper or lower culmination and meridian altitude observations of the sun for latitude are the methods used. The latter is used most extensively, a series of observations being taken on successive days. It is easily executed and appears as follows: Level the transit with the telescope in the meridian elevated to the sun's approximate altitude at noon; observe the altitude of the sun's lower limb with the sun slightly east of the meridian; reverse the transit; observe the altitude of the sun's upper limb with the sun slightly west of the meridian; and take the mean observed vertical angle for the altitude of the sun's center at apparent noon.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of Public Lands of the United States", 1919, p. 65.

The field record is as follows:* October 5th, 1909, in approximate latitude 37° 20′ N.; longitude 102° 04′ W.:

Setting:	90° 00′
$\phi \neq$ (—)	37° 20′ N.
$\delta \neq$ (—)	4° 42′ S.
$V \neq$	47° 58′
Lower limb.....	47° 42′
Upper limb.....	48° 14′

°+ Observed altitude, lower limb, direct	= 47° 43′ 00″
+° Observed altitude, upper limb, reversed	= 48° 16′ 30″
Mean observed altitude, V	= 47° 59′ 45″
Refraction —	0′ 52″
Parallax +	0′ 06″
h	= 47° 58′ 59″
δ	= 4° 41′ 42″ S.
$\phi = 37° 19.3′$ N. $= 90° - \delta - h$	= 37° 19′ 19″
	90° 00′ 00″

Meridian observations of the sun for time and latitude are conveniently combined, by observing simultaneously the sun's lower and west limbs, recording the watch time and the vertical angle, and reversing the transit in the interval of about 2 min., and then observing simultaneously the sun's upper and east limbs. Settings for approximate altitudes of the sun's lower and upper limbs are, respectively:

$$V \neq 90° - \phi \pm \delta \mp 16′$$

Azimuth.—There are a variety of methods and number of heavenly bodies from which to select for determinations of azimuth. A solar transit properly adjusted to the true meridian will serve best on line work in timbered country. In open country, however, it will often be found practicable to carry forward a transit line, always referred to the true meridian by deflection angles. The writer has frequently completed the resurvey of entire townships by deflecting the necessary angles from an initially determined stellar meridian, and when checking the last line run, against the true meridian, has intersected a spade handle set as a reference point. This may readily be accomplished by the careful handling of an instrument which is kept in perfect adjustment.

Methods of observation on Polaris and on the sun are unquestionably the most desirable for determinations of azimuth for the use of the cadastral engineer. Observations on Polaris at its eastern or western elongation, at any hour-angle, most conveniently taken at sunset or sunrise, and direct altitude observations and equal altitude observations of the sun are among those used. The latter method, however, has little adaptability to line work, but by virtue of the need only of approximations of time and latitude, the method possesses a certain usefulness in camp.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 67.

By far the most convenient and accurate azimuth determination is by observation of Polaris at any hour-angle, preferably during the daylight hours, thereby eliminating the necessity for artificial illumination and the personal equations of various assistants. However, with the direct rays of the sun above the horizon at sunset or sunrise, a close approximation of the position of Polaris is necessary before it may be found. The hour-angle, t, and azimuth, A, are ascertained in order to locate Polaris in azimuth. Its position in altitude is determined by the following approximation,* the positive sign being used for hour-angles of less than 6 hours and the negative sign for hour-angles exceeding 6 hours:

$$V \neq \text{Latitude} \pm (\text{Polar distance of star} \times \cos t) \neq \phi \pm 70' \cos t.$$

A computation* of the position of Polaris at sunset, May 6th, 1911, at a station in latitude 47° 20′ N., and longitude 102° 40′ W., appears somewhat as follows: The declination of the sun is found from the Ephemeris† to be 16° 18′ N., and the apparent time of sunset is found to be 7 hours 15 min., P. M., by reference to the Standard Field Tables.‡ Then the

			h. m.
Assumed time of observation, May 6th, 1911		=	7 15 P. M.
	h. m.		
Gr. U. C. of Polaris, May 6th	= 10 33.5 A. M.	+	12
Reduced to Long. 102° 40′ W.	— 1.1	=	10 32.4 A. M.
Assumed hour-angle of Polaris west of the meridian		=	8 42.6
Hour-angle, angular measure		=	130° 39′
Azimuth of Polaris, $\neq$ sin 130° 39′ × (azimuth at W. E.)		$\neq$	1° 17′ W.
Latitude of station	= 47° 20′		
70′ cos t = 70 cos 130° 39′ =	46 (—)		
$V \neq$	46° 34′		

In order to find Polaris an approximate meridian must be available as a reference from which to set off a horizontal angle of 1° 17′ W., and a vertical angle of 46° 34′.

In the twilight hours, after the passing of the direct rays of the sun below the horizon, only a rough approximation of the position of Polaris is necessary in order that it may be found, and there still remains enough daylight for clear identification of the reference mark and reading of the verniers. The writer has often determined the necessary co-ordinates of position roughly from a glance at a Hammet's planisphere set in position for the time and date. Although this method may not at first bring Polaris within the field of the telescope, by small quick movements of the telescope in departure or in latitude, the flicker of the star's light across the field will indicate its presence where a steady, direct sight will not. Especially is this the case when the pre-deter-

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 100.

† "Ephemeris of the Sun and Polaris, and Tables of Azimuths and Altitudes of Polaris", Commr. of General Land Office, pub. annually since 1910.

‡ "Tables and Formulas for the Use of U. S. Surveyors and Engineers on Public Land Surveys," ed. 2, Commr. of General Land Office, 1913.

mined sidereal focus of the telescope is found not to be true for use on the star. In this observation, a distant object is greatly to be preferred as a reference mark for reading horizontal angles, in order that this focus, when corrected for the star, will not have to be changed when sighting on the mark.

With a number of equations at the disposal of the engineer, to suit his convenience, a very efficient alternative for azimuth observations on Polaris is found in direct altitude observations on the sun. These observations are useful in that they can be taken on the line of survey during the day and used for the required tests of the solar attachment, which during appropriate hours for solar work is expected always to come within 1′ 30″ of the true meridian, before approval.

Under working conditions any line determined with the solar attachment may be used for reference purposes from which to obtain the necessary data for computing the true bearing thereof. In order to guard against error, a series of three altitude observations on the sun, each with the telescope in direct and reversed positions are required. These are readily obtained in 10 or 12 min., while the reductions may be made in the evening. When using this method of observation a full vertical circle, a colored glass shade in the dust shutter of the eyepiece, and a prismatic eyepiece are essential to rapidity of performance and accurate results. The standard improved instrument adopted for use on official surveys has this equipment.

The essential features of this observation consist in the simultaneous determination of the true vertical and horizontal angles to the sun's center. The relation between the calculated azimuth of the sun and the recorded angle to the sun's center gives the true bearing of the fixed reference line. Any one of the following equations* may be entered, and the azimuth of the sun's center as referred to the true meridian at the epoch of observation, may be calculated:

$$\tan \frac{1}{2} A = \sqrt{\frac{\cos \frac{1}{2} (\zeta + \phi + \delta) \sin \frac{1}{2} (\zeta + \phi - \delta)}{\cos \frac{1}{2} (\zeta - \phi - \delta) \sin \frac{1}{2} (\zeta - \phi + \delta)}} \quad \ldots\ldots(1)$$

$$\cos \frac{1}{2} A = \sqrt{\frac{\sin S \sin (S - \text{co-declination})}{\sin \text{co-latitude} \sin \text{co-altitude}}} \quad \ldots\ldots(2)$$

in which the "pole-zenith-sun" triangle is expressed as follows:

Pole to zenith $= 90° - \phi =$ co-latitude;
Pole to sun $= 90° - \delta =$ co-declination;
Zenith to sun $= 90° - h =$ co-altitude;
$S =$ one-half the sum of the three sides.

$$\cos A = \frac{\sin \delta}{\cos \phi \cos h} - \tan \phi \tan h \quad \ldots\ldots(3)$$

In case it is desired to obtain both time and azimuth, Equation (1) will be preferred. For a determination of azimuth only, Equation (3) will be found

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 104.

to be the most convenient on account of the comparative ease of its reduction. An example of direct altitude observation for azimuth on the sun, north declination, using Equation (3) will be given. For a south declination, the function, "sin δ", would become negative by virtue of the sine of a negative angle being treated as negative in analytical reductions; should the algebraic sign of the result be positive, the azimuth, A, is referred to the north point; if negative, A is referred to the south point.

Table 1 gives a series of three direct altitude observations on the sun for azimuth. These observations were taken on August 2d, 1909, at the corner of Ts. 31 and 32 S., Rs. 43 and 44 W., 6th Prin. Mer., Colorado, in latitude 37° 17.5′ N.; longitude 102° 18.6′ W., at 7:30 A. M. apparent time. From a meridian determined with the solar attachment, an angle of 90° was turned to the east. On this line, a flag was set about 20 chains distant, from which as a reference point these observations were taken.

TABLE 1.—Altitude Observations of the Sun for Azimuth.

Series of three observations.	Telescope.	Sun.	Watch time.	Vertical angle.	Horizontal angle from flag to sun.
1st.	Direct	∘+	7 h. 36 m. 54 s.	30° 05′	0° 08′ 30″ to N
"	Reversed	+∘	7 38 15	29 48	0 33 00 " "
	Mean			29° 56′ 30″	0° 20′ 45″ to N
2d.	Direct	∘+	7 h. 41 m. 20 s.	30° 58′ 00″	0° 32′ 00″ to S
"	Reversed	+∘	7 43 00	30 46 30	0 12 30 " "
	Mean			30° 52′ 15″	0° 22′ 15″ to S
3d	Direct	∘+	7 h. 52 m. 00 s.	33° 05′ 00″	2° 11′ 00″ to S
"	Reversed	+∘	7 53 48	32 53 30	1 50 00 " "
	Mean			32° 59′ 15″	2° 00′ 30″ to S

Reduce the mean observed vertical angle to the sun's center for each observation, to the true vertical angle, thus:

	1st Obsn.	2d Obsn.	3d Obsn.
V =	29° 56′ 30″	30° 52′ 15″	32° 59′ 15″
Refraction =	−1′ 40″	−1′ 36″	−1′ 28″
Parallax =	+ 8″	+ 8″	+ 8″
h =	29° 54′ 58″	30° 50′ 47″	32° 57′ 55″

Thence, reducing the first observation of the series for azimuth by Equation (3),

$$\cos A = \frac{\sin \delta}{\cos \phi \cos h} - \tan \phi \tan h.$$

The sun's declination for the mean period of three observations equals 17° 51′ 04″ N.

$\log \cos \phi = 9.900674$ $\log \sin \delta = 9.486493$ (+) $\log \tan \phi = 9.881708$
$\log \cos h = 9.937897$ $\log \tan h = 9.759970$

9.838571 9.838571 log $= 9.641678$

log 9.647922 nat (—) $= 0.43821$

nat (+) 0.44455
(—) 0.43821

$\cos A =$ (+) 0.00634

$A =$ true bearing of sun $=$ N. 89° 38′ 12″ E.
Angle from sun to flag $=$ (+) 0° 20′ 45″

True bearing of flag $=$ N. 89° 58′ 57″ E.

A similar reduction may be made of the second and third observations, for the true bearing of the sun, from which :

By first observation, flag bears N. 89° 58′ 57″ E.
By second observation, flag bears N. 89° 58′ 26″ E.
By third observation, flag bears N. 89° 58′ 38″ E.

Mean true bearing of flag N. 89° 58′ 40″ E.

Therefore, the indicated error of the solar attachment is 1′ 20.″

At any place in the field however remote, the cadastral engineer of the present day is seldom without the means by which to determine true values of time, latitude, and azimuth which are necessary to a correct return of his survey, nor does he often fail to surround his methods with adequate verification in order to insure their required accuracy.

IDENTIFICATION AND RESTORATION OF CORNERS

Permanency of Construction

The boundary lines of the original survey must be retraced and the true bearings and distances between identified original monuments determined before the limits of the areas covered by existing entries and patents may be identified and the computation of the remaining subdivisions of public lands may proceed. Lost or obliterated corners are restored to their true original positions by a proper reference to the original field-note record, influenced in some cases by competent collateral evidences which, before acceptance, however, must be justified in the light of the same record. Monuments restored in this manner, or those identified and reconstructed in place, no longer, as in former days, depend for permanency on shallow pits and mounds of earth with the mythical "charcoal deposit" or, in timbered and rocky regions, on small marked stones of indifferent composition and short-lived bearing trees.

When the true boundary is identified in a resurvey, after many retracements and much investigation, it is now possible to monument the corner permanently by virtue of the Act of Congress approved May 27th, 1908 (35 Stat., 347), and later Acts amplifying this authority. For this, a concrete-filled, 3-ft., iron corner post is used. It may be 1, 2, or 3 in. in diameter, and is set in

the ground with the fill tamped in over a broad flange at the bottom. If the soil is sandy and loose, the flange of the post is set in a concrete base. This post is then witnessed by the best of available accessories, such as marked bearing trees, pits, or mounds of stone. The corner position is carefully re-monumented without destroying the physical evidence which served to identify its original position, the utmost regard being shown for all such evidences of original location. A complete record is kept of the description of the original marks as identified and all the resurvey additions thereto. At the time they are used, the brass caps of the iron corner posts are suitably and plainly marked with steel dies, capital letters and Arabic figures being used. Fig. 5 shows a sample marking of brass caps. As an additional safeguard to insure the permanency of these monuments Congress, by an Act approved March 4th, 1909 (35 Stat., 1088, Sec. 57), has provided a fine and imprisonment for those who wilfully destroy them or effect their removal.

Recognition of Physical Evidence

The Courts attach major importance to authentic evidence relating to the original position of an official corner monument, such evidence being given far greater weight than the technical record relating to bearings and lengths of lines. The legal signficance of original monuments makes it mandatory on the engineer to exercise the greatest diligence to find and restore each corner in its true original position. On the treeless prairies of the Central Western States where original corners were generally constructed of wooden posts or small stakes with pits and mounds, the writer has identified and reconstructed hundreds of such monuments long obliterated and considered lost, by the ordinarily simple, but sometimes very expert, process of removing the top sod or soil with a sharpened spade and revealing the distinct outlines of the original pit beneath, in the exact form as left by the builder years before. The original pit has gradually filled with blowings and wash of a distinctly different nature from the homogeneous soil on all its sides. During many years of experience in this work, the writer has recovered indisputable outlines of such original pits under all conditions and kinds of soil—in roadways, on steep slopes, and on lands subject to overflow—and he has no hesitation in stating that when once dug if any part of the original sides of the pit still remains, the fill in the center can be identified with certainty by this method.

In timbered regions where Nature has healed the wounds made by the scribing tool of the original surveyor on the tree corner, or the "bearing" trees used as accessories to the corner, the expert will readily identify the new growth over the old blaze. A sharp axe applied above and below the growth will remove it in one piece, thereby uncovering the marks originally made on the tree itself and also those transferred to the piece removed, where they will appear raised in relief. Furthermore, removal of such growths from the old "line" trees and others of those which may mark the section line, will often reveal the old axe marks. This evidence, when corroborated by a correct "ring count" in the growth itself, will provide silent but convincing testimony

as to the position of the original line, even when the corner monuments themselves cannot be found. Illustrations of such growths, which were removed by the previously described method from the original blazes and on which the original marks appear in relief, will be found in Figs. 6 and 7. Fig. 6 shows a chip from a white fir tree which in the original survey of the boundary line between the States of Oregon and California was marked and described by Deputy Surveyor Major in 1868, for the 147th-mile corner. This fir tree was readily identified 47 years later—in 1915—by cadastral engineers in a resurvey of this line. The growths over the original blazes were removed, and the tree was found to be correctly marked, as follows: 147 M on the east; O on the north; C on the south; and 42 L on the west faces of the tree.

The chip shown in Fig. 6 is from the west face of this tree. It plainly shows the original marks, "42L", in relief, and is made up of forty-seven growth rings, corresponding to the interval in years which had elasped since the date of the original survey. In Fig. 7 will be seen a part of the growth removed from over the original marks on a cypress bearing tree at a meander corner on the line between Sections 32 and 33, T. 41 S., R. 37 E., Tallahassee Meridian, Florida. Originally, the thickness was a few inches greater than that shown in the photograph, the outside portion having been lost in the removal. The marks which plainly show in relief, were made on the tree in 1855, and the chip was removed by cadastral engineers in 1919. The "ring count" on the adjacent growth was "sixty-five", which was the correct time interval in years.

An interesting example of a restoration involving the identification of original bearing and line trees and the proper application of proportional measurement occurred in 1913, in the oil district of Louisiana, in the determination of the section and subdivision of section lines of Section 3, T. 20 N., R. 16 W., L. M. The methods used were thoroughly tested in the trial of a boundary suit where a difference of 1.7 ft. involved a property value exceeding $800 000. Many days were required in the trial to hear the technical evidence alone. The following extract from the decision of the Supreme Court of Louisiana (No. 22 485, February 25th, 1918), is illustrative of the severe requirements of such restorations:

"Among the several surveys made, one stands out as most worthy of consideration by the Court. It was not made on behalf of any of the parties interested in this litigation, but it was ordered by the United States Government and it was executed under instructions from the General Land Office, whose stamp of approval has been placed upon it. Under these circumstances, the recognized ability and competency of the engineer, the total absence of any possible bias on his part, the great care he exercised in the performance of his work, the most modern and scientific methods adopted by him and the further fact that the result of his work bears the approval of the General Land Office, are, in our opinion, sufficient to establish a preponderance of evidence in favor of plaintiffs and to justify a decree based upon his findings under the law applicable in the case."

Methods of Restoration

The original corners restored in a resurvey may be divided into two general classes: (*A*) those identified and reconstructed in place; and (*B*) those

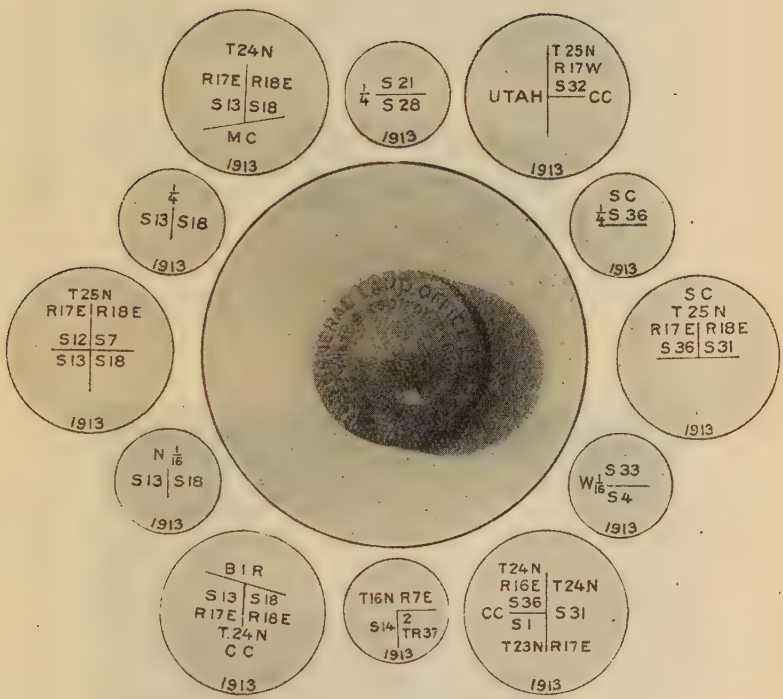


FIG. 5.—SAMPLE MARKINGS OF BRASS CAPS.

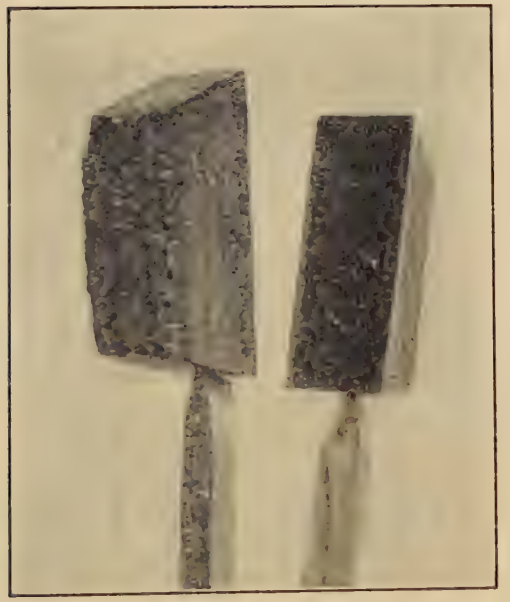


Fig. 6.—Overgrowth Taken from White Fir Tree Corner on Oregon-California State Boundary.

Fig. 7.—Overgrowth Taken from Cypress Bearing Tree in Florida.

restored by a mathematical or ethical procedure. Further defined, they will appear as follows:

(*A*).—A corner is identified and may be reconstructed in place:

1.—When the physical monument itself or any of its accessories are found and identified in their original positions.

2.—When the physical witness corner or any of its accessories are identified in position, from which the true point for the corner is determined.

3.—When, although no physical trace of the monument or accessories remains, its original position has been perpetuated with certainty by settlers.

Referring to Paragraph 2, it naturally follows that, as the true point for the corner was not originally monumented, it is only necessary in such a case to identify the original witness monument in its original position and then proceed to the true corner point in accordance with the original record.

Identification under Paragraph 3 includes those instances where the original corner had been found at one time and its true position perpetuated by some permanent mark of local origin, after which the physical evidences of the official monument were destroyed, and, only for the permanent mark, the original position would have been lost.

(*B*).—A corner is identified and may be restored to place:

1.—When an appropriate reference is made to the original record, from which is established a definite relative position with respect to other adjacent and existing competent points of control, as follows: (*a*) other existing public land corners; or (*b*) corners of other systems of surveys or unchanged and definite original points of topography.

2.—When a position has been recognized for years as the location of the corner by all who reside in the vicinity, and has been agreed on and perpetuated as such by the several parties in interest; such point, however, must hold an acceptable relative position when compared with those original official monuments within the field of influence of which the point is found to be situated.

The method of Paragraph 2 under Heading, (*B*), does not imply necessarily that such position has any direct relation whatever to the true position of the corresponding original monument itself. Unusual cases will also arise where the nearest existing original corners are so far distant (all intervening monuments having been lost) as to lose in some measure their influence and value as factors of control. In this event, other competent collateral evidence will correspondingly increase in relative value for the identification of an acceptable position for the corner.

A choice must always be made between the several methods of restoration. The appropriate method will depend, of course, on local conditions existing in the particular case under consideration, selected so as to afford an equal protection for all the interests involved. To accomplish this result, it does not always follow that the most probable position of the lost original monument

will be the one accepted when a proper consideration and combination of all available evidence is made. When the physical corner itself has once been destroyed and the original position lost, any method selected by which to restore it to its most probable position will not do so with certainty. It is better at once to adopt the position with respect to the nearest existing corners which has the greatest possible concordance with the original record, as all disposals of public lands are made in accordance with the original plat representing such record, both as to the position and extent of land intended to be conveyed. This is exemplified in the method ordinarily proper for the restoration of closing corners in cases where the closing line defines the boundaries of alienated lands. In such cases, the identification may be made by means of a proportional measurement based on the plat distances to the right and left of the closing corner, measured along the line closed on, rather than on the single record connecting tie of the original surveyor alone; also, the same reasoning applies in the identification of those lost corners immediately preceding on the closing line, namely, the lost corners on a range line closing on a standard parallel, the identified positions of which ordinarily are based on a proportional measurement extending to the major line closed on (standard parallel), and not to the original closing monument itself, in cases where the monument is known and is off the major line.

Proportional Measurement

The method given under Heading (*B*), Paragraph 1, is, in general, that method ordinarily referred to as "proportional measurement", by which is meant a measurement having the same ratio to that recorded in the original field notes as the length of the line by re-measurement bears to its length as given in the record. The ideal condition, of course, would be that in which the corners chosen as a basis for control would occupy in all directions the position they should with respect to each other without any excess or deficiency; however, such is rarely, if ever, the case. The evidences of position offered by these controls are always conflicting to a greater or less extent, but as they are of the same type and as controlling factors vary inversely as the distances involved, these evidences, although conflicting, may be consistently combined and, at the same time, admit of a practical and more or less general application. This has resulted in what is called the single and double proportionate method, on which, it has been found, dependence must be placed in the execution of resurveys for the major number of restorations, and with respect to which the only general rule of procedure has been formulated.

An exception is made, however, to any such general rule of proportion in those cases where, on account of the existence of gross error and irrelation in the evidences of the original survey, or, in the presence of extensive obliteration, only one or only two of the nearest existing original corners have been used by entrymen for the control of locations where more controlling points should technically have been considered. The location of such a claim, so determined and perpetuated by improvements, will often differ materially from that determined by a proper consideration of all the existing corners, that is,

two or four. In such cases, it cannot generally be charged that a lack of good faith has been shown, and those acceptable positions for the lost corners as fixed by evidences of occupation are then adopted for the identification of the tract and possibly for other tracts which may be in harmony therewith. In these cases of irrelation, natural conflicts will often be found to exist and must be recognized and identified on the ground by the engineer for adjudication by the proper Court. These locations are best identified in the form of tract segregations by metes and bounds in "independent" resurveys, further consideration of which will follow.

Theory of Non-Concurrent Parallel Forces

On all lines of the same class in any survey, each call of the original record should bear, theoretically, its proportion of responsibility for discrepancies found to exist between evidences of such survey on the ground and the record thereof, and the errors should vary directly as the distance, along any line of such survey, of an identified control away from the locus of the lost corner. For example, if five or more monuments had been established originally, one of which is assumed to be lost, its restoration might be based on a consideration of as many temporary points as there are existing monuments, each temporary point to be assigned its proper value or weight and to be located by proceeding on the record course and distance from one of the existing monuments. The temporary points thus determined might then be treated as a system of non-concurrent parallel forces acting normal to a horizontal plane, the position of the resultant of which is sought for the position of the lost corner. The restoration would be accomplished by assigning to each temporary point a value inversely proportional to its distance from the initial existing monument at the other end of the random line. Obviously, this academic reasoning could be applied in any practical restoration of public land corners only as between the nearest identified corners in any direction, in order to avoid a needless multiplicity of temporary points. If it could not be assumed that those corners which are situated outside of a certain restricted field of influence, are at so great a distance away as to have little or no effect, it at once becomes evident that a strict adherence to the foregoing principles would have no value for a practical application.

Fig. 8 illustrates the theory of non-concurrent parallel forces, as applied from a four-point control, for example:

Let *A, B, C,* and *D* represent four identified bearing trees witnessing the position of a corner of the original survey; also, let *a, b, c,* and *d,* represent four corresponding temporary points for the restored corner, the positions of which are determined by the courses and lengths given in the original record as run from the identified original bearing trees, *A, B, C,* and *D.*

Since *C* and *D* are distant from *c* and *d* only one-half the distances of *A* and *B* from *a* and *b,* the temporary points *c* and *d* will be assigned a measure of influence (or weight) twice that of *a* and *b.*

Therefore, with *a* and *b* each valued as 1, *c* and *d* will each be rated as 2. As the values of *c* and *d* are equal, which is also true of *a* and *b,* the combined influence of each pair may be represented, as to *c* and *d* by

a temporary point, *e*, mid-distant on Line *cd*, with a weight of 4, namely ($e = c + d = 2 + 2 = 4$), and as to *a* and *b* by a point, *f*, mid-distant on Line *ab*, with a weight of 2, namely ($f = a + b = 1 + 1 = 2$). Thence, the true point for the restored corner which results, with due regard for the influence of each bearing tree, will lie on a line connecting *e* and *f*, a distance from *f* to *e* equal to two-thirds of the total, namely $\left(\frac{4}{2+4} = \frac{2}{3}\right)$.

The resultant point on the line, *ef*, will be removed, therefore, from *e* or *f* distances which assume an inverse ratio to the respective weights assigned thereto. Any combination of the points, *a*, *b*, *c*, and *d*, with properly assigned weights, will give the same resultant position for the true corner.

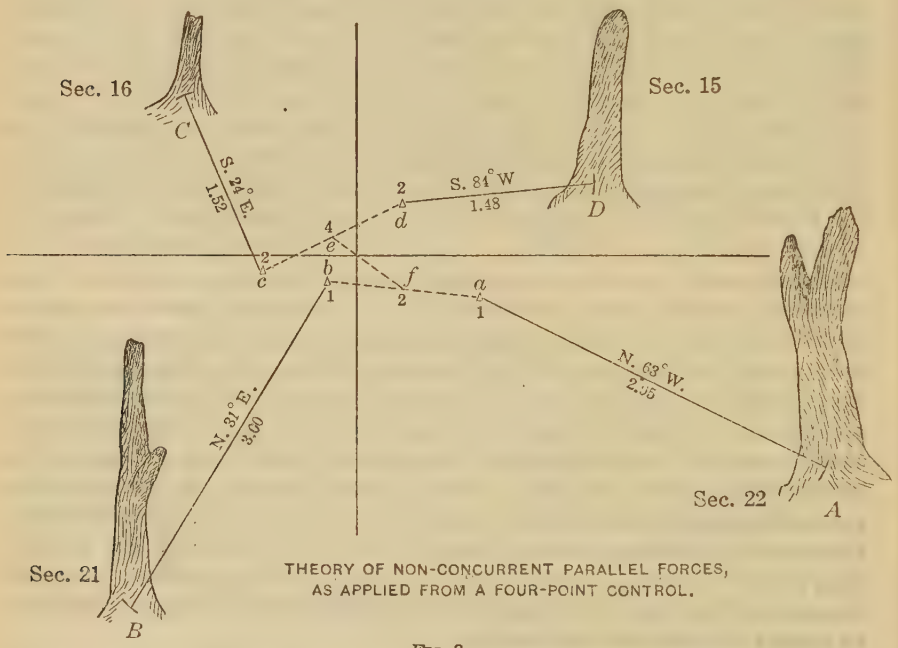


FIG. 8.

Application of Theory to Broken Boundaries.—The restoration on the ground of any angle-point of a meander line, between the two nearest meander corners, would appear in the execution somewhat as shown in Fig. 9.

Let *A* and *B* represent, respectively, a re-established and a reconstructed original meander corner on the east and west boundaries of Section 28. The numbered lines, 1, 2, 3, and 4, and 6 and 5 represent consecutive meanders run out from *B* and *A*, respectively, on the record course and distance, to arrive at the temporary points, *b* and *a*. Then, the restored position for the desired angle-point will be on the line, *ab*, a distance from *b* represented by the fraction,

$$\frac{1+2+3+4}{1+2+3+4+5+6}$$

in which the numerals represent the record lengths only of the correspondingly numbered meander lines.

Similarly, all the angle-points of such a line may be reproduced on the ground by running out the complete record from either B or A, as shown graphically in Fig. 10.

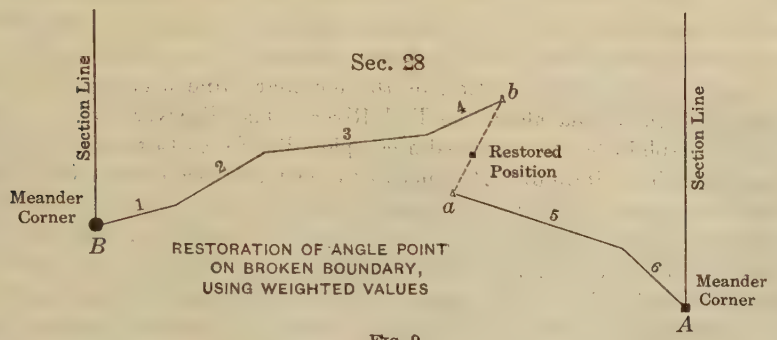


FIG. 9.

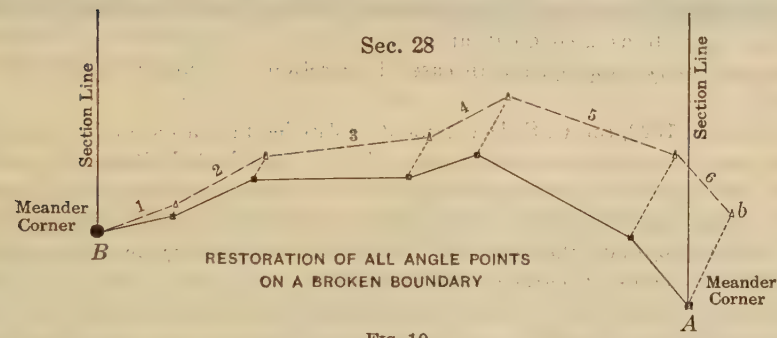


FIG. 10.

In Fig. 10, the closing error, Ab, is by construction equal to the length, ab, in Fig. 9. To effect a restoration of the entire meander line in Fig. 10, lay off from each temporary angle-point in the direction and on a line parallel to Ab, a distance which will be a proportional amount of the length of the closing error. This distance will be that one which is to the whole length of the error (Ab) as the distance of the particular temporary point from the initial point (B) is to the whole length of the meander line (from B to b). If one or both of the meander corners on the section lines are lost, they are first restored, before proceeding with the reproduction of the meander line.

Lost corners originally established in metes and bounds surveys, and all other irregular surveys not of the regular rectangular system, broken controlling boundaries, and the angle-points of a meander line, are restored and reproduced on the ground according to the principles as described. A valuable discussion* by Leonard S. Smith, M. Am. Soc. C. E., of a similar method of restoration of a broken boundary in which the problem was the retracement

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 429.

of an old road, will be found to be of great interest to those in contact with this subject. In this case, the "twist" or error in azimuth of the original surveyor's meridian was computed and also the ratio of the lengths of the original and the resurvey chains.

Oklahoma-Texas Boundary Controversy.—A very important boundary dispute of related interest, now in litigation, and one in which the U. S. Cadastral Engineering Service is making extensive cadastral and topographic surveys for the preparation of the necessary maps, is that which involves the determination of the true location of the boundary line between the States of Texas and Oklahoma along the Red River. The discovery of oil within the river flood-plain has precipitated a complex dispute as to the ownership of the river bed. The State of Oklahoma in the October Term, 1919, filed a suit in equity, Original No. 27, in the Supreme Court of the United States *versus* the State of Texas, Defendant. The question at issue involves the construction of the Treaty of 1819, between the United States and Spain, as to whether the specifications of the treaty refer to the south bank of the Red River or the middle thread of the main channel—the treaty, of course, referring only to the river in the position it occupied in 1819. In the determination of this position, the comparatively new science of Ecology* will have an important bearing, as well as geological and topographical considerations. Riparian rights such as are appropriate in case of accretion or avulsion might then be applied.

In March, 1920, the U. S. Department of Justice filed a motion to intervene in the suit, in order to conserve the rights of the United States to possible public domain and to protect the rights of the Indian wards of the Government within the river basin. The Supreme Court, in April, 1920, granted this motion, enjoined the State of Texas from selling any purported rights covering any lands or parts of the river bed lying north of the line of the south bank as it existed in 1819, and appointed a receiver to take over the operation of the oil field in certain ranges and between the south bank and middle thread of the main channel of the river. A suit was subsequently filed in the U. S. Federal Court for the Western District of Oklahoma, by the Department of Justice, in behalf of the Indian allottees, in order to protect riparian claims attached to Indian allotments on the north bank and extending to the medial line between the river banks (the present meander lines of the right and left banks), which according to the Indian treaty of 1867, is the line designated as the south boundary of the Kiowa and Comanche Reservations. The District Court has appointed a receiver to take over the operation of the oil field within the riparian claims of the Indians. The receiver for the Supreme Court under a working agreement has control of any areas in conflict between the two receiverships.† Separate consideration will be given in these suits to the questions of fact as to the positions determined for the true boundaries at the definite dates. A reference to Figs. 11 and 12 will give the reader a very good idea of the appearance

* Plant Ecology: the Adaptation of Plants to Life Under Particular Environing Conditions; one of the chief exponents of the science in the United States is Dr. Henry C. Cowles of the University of Chicago.

† Report of the Commr. of the General Land Office to the Secretary of the Interior, June 30th, 1920, pp. 33–34.

FIG. 11.—RED RIVER, LOOKING NORTH FROM TEXAS SIDE. IRREGULAR FULL LINE INDICATES POSSIBLE POSITION OF 1819 RIVER BANK. DASHED LINE SHOWS SOUTHERN LIMIT OF AREA UNDER SUPREME COURT RECEIVERSHIP.

FIG. 12.—RED RIVER, LOOKING NORTHEAST FROM TEXAS SIDE. FULL LINE INDICATES POSSIBLE POSITION OF 1819 RIVER BANK. DASHED LINE SHOWS SOUTHERN LIMIT OF AREA UNDER SUPREME COURT RECEIVERSHIP.

of the oil field under the receivership. Fig. 11 is a view of the Red River, south of T. 5 S., R. 14 W., I. M., Oklahoma, looking north from the Texas side. The irregular full line indicates the possible position of the 1819 river bank, and the dashed line shows the southern limit of the area under the Supreme Court Receivership. Fig. 12 is a view of the Red River, south of T. 5 S., R. 14 W., I. M., Oklahoma, looking northeast from the Texas side. The full line indicates the possible position of the 1819 river bank. The dashed line shows the southern limit of the area under the Supreme Court Receivership. As the bed and flood-plain of the river are located over an oil "dome" or pool, nearly every well has been a producer and property values exceeding $100 000 000 are involved in this boundary litigation. The photographs were taken during 1920.

Erroneous and Fraudulent Meander Lines.—Meander lines as shown on approved plats are sometimes erroneous or fraudulent. In these cases, the lines as recorded do not and never did conform even approximately to the mean high-water elevation of the actual body of water, and, thus becoming fixed boundaries, they sometimes omit large areas of public land from the original surveys. The location of a true mean high-water mark as of an early date, for critical comparison with the restored record position of the original meander line, overlaps somewhat into the realms of geology and ecology. These questions while involving the tracing of old escarpments in the soil and investigations into the age and growth of trees, are not overlooked by the cadastral engineer.

In the practical application of the foregoing mathematical method of restoration, which contemplates the use of both lengths and courses of original lines and utilizes weighted values, to the cardinal lines of the rectangular system of original surveys, the necessary introduction of three or four temporary points, on which to base a final determination of a restored position, complicates the computations and the record of the field operations to such an extent that the small advantage which the method possesses as to exactitude ordinarily does not justify its use in preference to the more adaptable and less complicated single and double proportionate methods of restoration. Except in cases where excessive distortion is developed, all restorations along the lines of the rectangular system will yield readily and logically to the principles involved in the latter methods. Although both the "weighted value" and the "double proportionate" methods depend on the same general underlying principles, the latter, although less theoretical, will produce results equally as good and is more practical of application than the former method, and has been adopted for official use.

Single and Double Proportionate Methods

The single and double proportionate methods of restoration are based on a consideration of the record distances only, the record courses in all cases being ignored, the measured distance between the two nearest existing corners being divided into distances proportional to those given in the record between the same corners. A necessary advantage of this method over that of the "weighted values" is that the restoration of one lost corner may be based on

the restored position of any other lost corner without producing the incongruities which will develop occasionally in using the method by "weighted values".

In explanation of the "double proportionate" method as now adopted for official use, assume that the problem is the restoration, from a four-point original control, of a lost section corner on the rectangular subdivisional lines of a township survey. A retracement is first made on the meridional line connecting two of the original corners, on which line a temporary stake is set at the proper proportionate distance, based on the original record. This point will determine the latitude of the lost corner. Next, the two original corners remaining are connected by a latitudinal retracement and a corresponding temporary stake set thereon, determined by proportionate measurement in a similar manner. The second point determines the position of the lost corner in departure. The intersection of two cardinal offset lines run from these two temporary stakes, north, south, east, or west, as relative situations may determine, will then define the restored position of the lost corner. The obliquity of the section lines in their restored positions, such as occasionally results, suggested some years ago the application of a slightly different method of proportionment based on those field distances which are measured along the hypotenuses of the several right-angled triangles created instead of along the long legs. This method when utilized provides an interesting and intricate mathematical study and, in the past, has required of the writer many days of harrowing field computations for its application. However, the impracticability of this method for general use is now admitted, inasmuch as the time-consuming field operations, extra computations, and complications of the record have proven wholly incommensurate with the doubtful degree in equity attained. Restorations are designated as "single proportionate" when made on lines established with reference to a definite alignment in one direction only.

IDENTIFICATION OF ALIENATED LANDS

General Principles

The fundamental principles used for the protection of *bona-fide* rights under the law are identical in all types of resurveys. In the case of the identification of the original position of any tract of land to which private rights have attached under the original survey, the identification when completed is regarded as a demonstration on the part of the General Land Office, in the light of the best evidence available, of the original position of those entered or patented subdivisions which are included in the description of the entry or patent, all as referred to the original survey. The cadastral resurvey, as far as private rights are concerned, has no creative function of its own, and it cannot change or supersede a completed function of any monument of the original survey. It must accept, without change, these original evidences of location as they are found to exist. Before a settler who "squats" or establishes residence on unsurveyed land, may make entry in accordance with

the legal subdivisions of an official plat after the land has been surveyed, he is required to adjust his lands to the descriptions and lines of the official survey. The settler who establishes himself on lands already subdivided and platted has the advantage of the "squatter", in that he has the original corners and an original record at his disposal to facilitate the proper location of his lands. Naturally, no less is required of the entryman on surveyed lands, who has monuments and an original record to guide him, than is required of the "squatter" on unsurveyed lands who has none; for the lands of the former are also required to bear an acceptable harmonious relation to authentic evidences of the original survey, in accordance with which his entry was made. For example, if a settler makes entry for lands described in Section 5 and then establishes his residence and erects improvements not on Section 5, but somewhere on Section 7 or Section 17, his location is clearly erroneous under his original description, and, therefore, the description of his lands must be changed if he is to obtain a clear title to those legal subdivisions actually occupied. If a patent has been issued under the erroneous description, a corrected patent may be issued on a proper showing in lieu of the original. This may only be accomplished in accordance with the regulations issued under authority of the legislation providing for amendment of entries (Act of February 24th, 1909, 35 Stat., 645).

Bona-Fide Rights

Thus, the cadastral engineer of the Government draws a clear distinction between the *bona-fide* rights as to location and as to occupation. Only with the former is he directly concerned, for the rights as to occupation are without his province and are entirely within the jurisdiction of the adjudicators of the proper court of law. If an entryman finds that his lands are erroneously located, his relief may be found only in accordance with existing law and not in any particular inspiration of a surveyor. However, in the case of irrelation and obliteration of corners of the original survey, where any one of several possible original positions is acceptable, that acceptable position is used which more nearly than any other covers the lands actually occupied and improved; in other words, the improvements are taken to indicate the particular original corner or corners which are to be used to control the position of the lands. The definition of good faith in location, as furnished the cadastral engineer by the Commissioner of the General Land Office, is necessarily broad, as witness the following:*

"It may be held generally that an entryman has located his lands in good faith * * * when it is evident that his interpretation of the record of the original survey as related to the nearest existing corners at the time the lands were located (as defined by his fencing, culture, or other improvements) is indicative of such a degree of care and diligence upon his part, or that of his surveyor, in the ascertainment of his boundaries, as might be expected in the exercise of ordinary intelligence under existing conditions."

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States," Commissioner of General Land Office, June 16th, 1919, p. 278.

In the application of this definition to field conditions, the U. S. Cadastral Engineer has become properly impressed with the purpose of his task and the stability and dignity which is attached to a work so great and important, commensurate with the broad foundation in science and law. The records of a cadastral resurvey must form an enduring basis on which depends the security of the title to all lands acquired thereunder, and the field notes must be prepared so that under the test of the closest possible scrutiny at all times, present and future, the record can be regarded as conclusive in the matter of the proper location of private rights.

General Types of Resurvey

In general, any field condition that may arise will yield properly to the application of either of two recognized methods of executing Government resurveys, namely, the dependent or independent resurvey.

The Dependent Resurvey.—The typical dependent resurvey may be defined as an official re-marking on the ground of all the original section lines in their original positions, in accordance with the best available evidences thereof, in such a manner that all original subdivisional units, as identified, will occupy those positions which result from a proper legal subdivision of the sections thus restored. It is obvious that this type of resurvey is chiefly applicable to those cases showing a fairly concordant relation between conditions on the ground and the record of the original survey, for titles, legal areas, and descriptions are maintained as unchanged. The primary control for such a resurvey is based, first, on identified existing corners of the original survey and other acceptable points of control; and, second, on the restoration of missing corners by proportionate measurement in harmony with the original record.

The Independent Resurvey.—On the other hand, in areas where gross error or irrelation is present, as between the existing evidences of the original survey, a zone of uncertainty often exists wherein the strict application of proportional measurement is either impossible or entirely inadequate for the protection of rights acquired in good faith under the law. In such cases, segregation of private land claims by metes and bounds surveys with a suitable tract number, from a one-point control according to the original record, is often necessary, or the conformation of such claims to the lines of an independent resurvey under either the original or new legal subdivisions, as may be appropriate; in either case, however, the claim as identified will occupy an acceptable position when referred to suitable evidences of the original survey. In such cases, there is generally no necessity whatever of applying any original restorative process for the description of the remaining unclaimed public domain, and an independent resurvey is executed in accordance with the cardinal lines of the official rectangular method of subdivision which, as to the vacant public domain, then supersedes the record of the original survey. In this type of resurvey, in cases where original descriptions are changed, the descriptions under the resurvey (tract segregations, or legal sectional subdivisions) and under the original survey of all private claims are appropriately shown, cross-referenced in an index table on the plat, and, in all

cases, existing private claims are blocked out in place and designated on the plat for further identification.

With respect to the protection of *bona-fide* rights, the basic principle is identical in either the dependent or independent type. In the former, all the lands patented and entered are identified theoretically by the legal subdivisions of the exhibited sections; in the latter, the identification is accomplished either by tract segregations or by conformations of the private claims to the sectional subdivisions, under either the old or new descriptions.

On Plate V is shown the resurvey of T. 27 N., R. 40 W., 6th P. M., Nebraska, which embraces boundary lines exemplifying both the dependent and independent types of resurvey, the limiting boundary between the two being along the original eastern boundaries of Sections 3, 10, 14, 23, 26, and 35, and the northern boundary of Section 14. The sectional lines in the dependent portion are covered by a resurvey, as shown on the plat approved October 4th, 1883, in their original positions according to the best available evidence of the positions of the original corners; all differences between the measurements shown on the original plat and those derived in the retracements have been distributed proportionately between accepted corners in accordance with surveying rules. The sectional lines shown in the independent part are not established on cardinal courses with regular corner spacing as is generally the case when this method is utilized, for by reason of a condition peculiar to this locality they were retained as delineated in the field, on a basis of proportional measurement against the "limiting" boundary. The plat does furnish, however, a normal example of the conformation of several entries to the lines of the resurvey under their original descriptions, and also exhibits various metes and bounds surveys which have been executed by the use of a one or two-point control. In Table 2 will be found the necessary information as to the segregated tracts, and in Table 3 areas of the various conflicts and exclusions are given. The method shown of treating the areas in conflict provides a comprehensive basis for subsequent adjudication of the private rights involved, whether to all or any part of the lands in controversy. In the particular locality under discussion, settlement of the litigation resulting from the strife of the several boundary disputes was suspended by the local Court until the plats of the resurvey became available.

TABLE 2.—PLAT OF RESURVEY IN NEBRASKA SEGREGATED TRACTS.

Resurvey.	Kind.	Entry.	Entryman.	Description under Original Survey.	Sec.	T. N.	R. W.	Status.
Tract 99...	H. E.	04623	Jacob W. Neumayer	S½. Sec. 31, T 28 N., R. 39 W., and N½	6	27	39	Patented
Tract 37...	H. E.	03890	Hugo Hein	All of	1	27	40	Patented
Tract 38...	H. E.	030	Charles S. Winston	S½, Sec. 6, and N½	7	27	39	Patented
Tract 39...	H. E.	07613	Frank Kingsbury	S½, Sec. 7, and SW¼	8	27	39	Patented
Tract 40...	H. E.	06826	Floyd L. Downing	All of	12	27	40	Patented
Tract 43...	H. E.	04908	Israel Miller	All of	18	27	39	Patented
Tract 44...	H. E.	06643	Luke M. Shultz	All of	19	27	39	Patented

TABLE 3.—Plat of Resurvey in Nebraska, Areas of Conflicts and Exclusions.

	In Conflict with:	Area, in acres.		Exclusive of Conflict with:	Area, in acres.
Tract 37...	Sec. 36, T. 28 N., R. 40 W.	80.18	Tract 37	Sec. 36, T. 28 N., R. 40 W.	560.68
Tract 37...	Indem. Sel. List 2, S½, Sec. 35, T. 28 N., R. 40 W.	84.80	Tract 37	Indem. Sel. List 2, S½, Sec. 35, T. 28 N., R. 40 W.	556.06
Tract 37...	Sec. 2, T. 27 N., R 40 W.	263.22	Tract 37	Sec. 2, T. 27 N., R. 40 W.	377.64
Tract 37...	All lands shown above	428.20	Tract 37	All land shown above	212.66
Sec. 2.....	Tract 37	263.22	Sec. 2	Tract 37	394.60
Sec. 2.....	Tract 40	88.90	Sec. 2	Tract 40	568.92
Sec. 2.....	Tracts 37 and 40	352.12	Sec. 2	Tracts 37 and 40	305.70
Tract 40...	Sec. 2	88.90	Tract 40	Sec. 2	557.70
Tract 40...	H.E.06123 as to Sec. 11	252.60	Tract 40	H. E. 06123 as to Sec. 11	394.00
Tract 40...	Sec. 2 and H.E. 06123	341.50	Tract 40	Sec. 2 and H. E. 06123	305.10
H.E.06123.	Tract 40	252.60	H.E.06123	Tract 40 (as to Sec. 11 only)	238.38
Tract 43...	Sec. 13. T. 27 N., R. 40 W.	313.12	Tract 43	Sec. 13	317.18
Sec. 13....	Tract 43	313.12	Sec. 13	Tract 43	368.88
Sec. 13....	Tract 44	69.72	Sec. 13	Tract 44	612.28
Sec. 13....	Tracts 43 and 44	382.84	Sec. 13	Tracts 43 and 44	299.16
Tract 44...	Sec. 13	69.72	Tract 44	Sec. 13	570.36
Tract 44...	Sec. 24	347.91	Tract 44	Sec. 24	292.17
Tract 44...	Secs. 13 and 24	417.63	Tract 44	Secs. 13 and 24	222.45
Sec. 24....	Tract 44	347.91	Sec. 24	Tract 44	385.29

Jurisdiction

The extent of the jurisdiction and authority of the Cadastral Engineering Service through the Commissioner of the General Land Office should be clearly understood. Section 453 of the Revised Statutes, reads, in part, as follows:

"The Commissioner of the General Land Office shall perform, under the direction of the Secretary of the Interior, all executive duties appertaining to the surveying and sale of the public lands of the United States, or in any wise respecting such public lands; and, also, such as relate to private claims of lands, and the issuing of patents for all grants of lands under the authority of the Government."

The Act approved March 3d, 1899 (30 Stat., 1097), re-affirms:

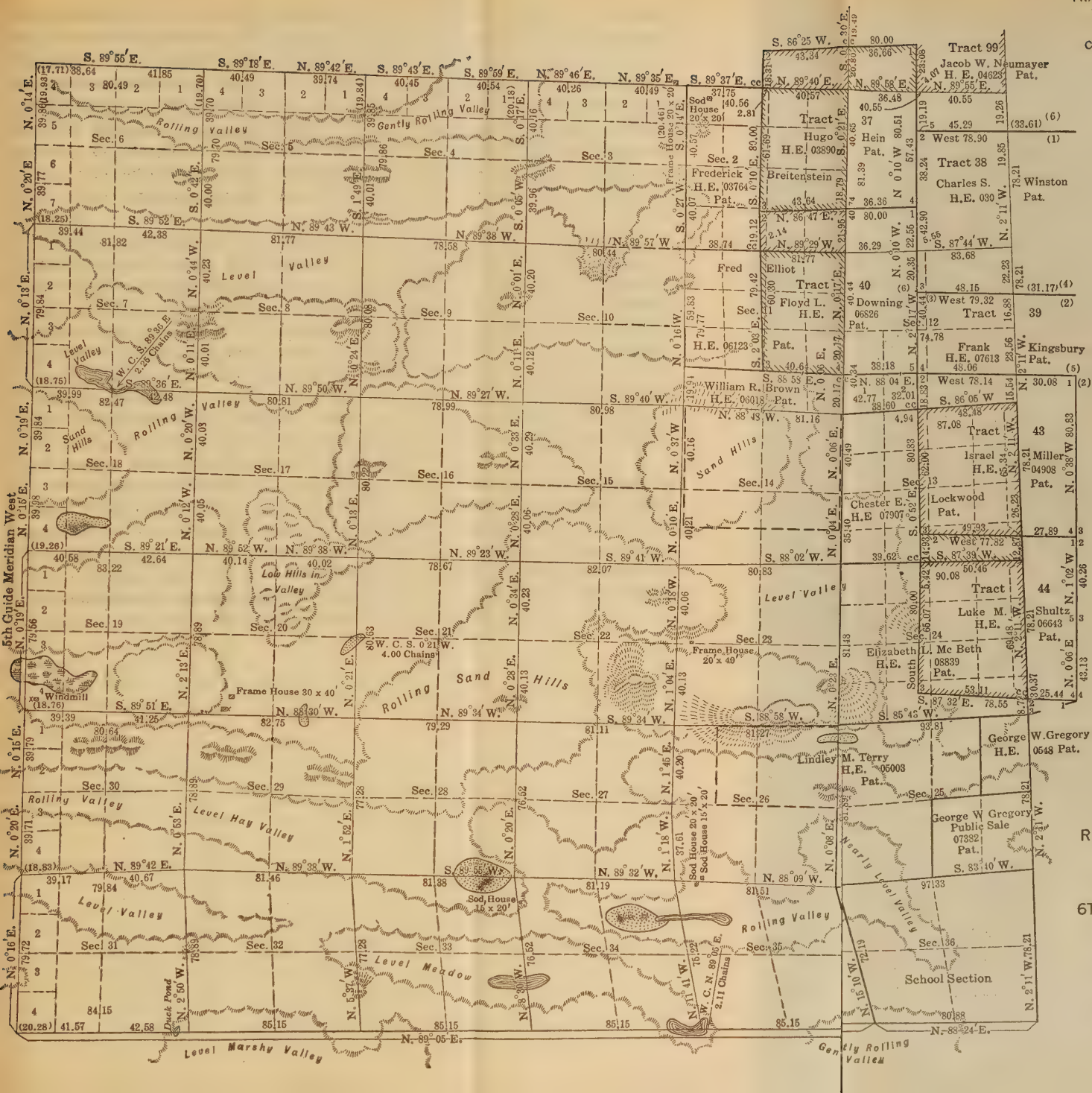
"That hereafter all standard, meander, township, and section lines of the public land surveys shall, as heretofore, be established under the direction and supervision of the Commissioner of the General Land Office, whether the lands to be surveyed are within or without reservations, * * *."

The Acts of March 3d, 1909 (35 Stat., 845), and of September 21st, 1918 (40 Stat., 965), provide in part:

"That no such resurvey or retracement shall be so executed as to impair the *bona-fide* rights or claims of any claimant, entryman, or owner of lands affected by such resurvey or retracement."

From this it is clear that, in the exercise of his authority, the Commissioner of the General Land Office shall cause all resurveys to be executed in such a manner as to afford a fair and equitable identification of the original boundaries of all private claims, so that rights acquired in good faith shall be unimpaired. If title has not finally passed from the Government, the Commissioner is clothed with the authority to perform any and all acts incident, appertaining to, or necessary for, the disposal of the lands. Where a patent has been issued,

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXV, No. 1489.
FARNSWORTH ON
CADASTRAL RESURVEYS.



RESURVEY OF TOWNSHIP
NO. 27 NORTH
RANGE NO. 40 WEST
6TH PRINCIPAL MERIDIAN,
NEBRASKA

the jurisdiction of the Land Department over the lands patented terminates, and its power to examine and decide on claims to such lands becomes exhausted. *Bona-fide* rights, however, can be affected by a resurvey only in the matter of position or location on the earth's surface, and the cadastral engineer is concerned only with the question as to whether lands covered by such rights have actually been located in good faith. Other questions of good faith, such as priority of occupation, possession, continuous residence, value of improvements, and cultivation, when considered apart from the question of the position of the original survey, do not affect in any manner the problem of resurvey.

The Commissioner of the General Land Office is well qualified, and is authorized, to identify and indicate the original positions of all lands, as described in outstanding patents, and to delineate the boundaries of the public lands remaining, irrespective of whether the lands thus identified are actually occupied by a patentee or his successor in interest. However, in case of either patented or unpatented lands, where an owner or entryman seeks to maintain a claim that his rights as to location have been impaired by a resurvey, he will submit a proper protest, and the case will still remain within the jurisdiction of the General Land Office (subject, however, to the right of appeal to the Secretary of the Interior), which office after a proper consideration of the protest will reach a final adjudication of the dispute.

It will thus be realized that, by virtue of the legislation now existing, in any and all areas where uncertainties of the true locations of lands obtain, especially in those regions ridden with boundary disputes, strife, and litigation, the application of the modern resurvey procedure and the subsequent filing of a competent plat, will provide a necessary and enduring basis, not only for a comprehensive adjudication of the private rights involved, but also for the disposal of any public lands which may be found therein.

DISCUSSION

E. M. Douglas,* M. Am. Soc. C. E. (by letter).—It is to be regretted that the author did not include in his paper a description in greater detail of the latest methods of making original cadastral surveys as well as resurveys. The writer's experience leads him to believe that all survey work by the General Land Office since the contract system was discontinued, has been well done by methods which may be accepted as models for other surveyors.

The writer is particularly interested in the references to the methods for determining true azimuth. The desirability of reporting true bearings should be impressed on every engineer who has long lines to run for cadastral surveys, boundaries, railroads, highways, or for public works of any kind. Too many of such surveys have been based on magnetic bearings, with no clue to the amount of the local variation of the needle, an oversight which produces much confusion and uncertainty when retracements are required. If engineers could be made to realize how easy it is to determine true azimuth by observations on the sun or on Polaris, more of them would use such methods. In reasonably clear weather, Polaris can be seen at any hour of the day with a good transit telescope.

To find Polaris in broad daylight, three things are essential:

1.—The telescope must be accurately focused. The proper focus can be found by observing the star at night and marking the telescope draw tube for future reference.

2.—The star's apparent altitude must be known within about 5 min. of arc, which can be found from tables. The U. S. Geological Survey will send such a table free, on request.

3.—The true bearing of Polaris at the time of observation must be known within less than half a degree.

The angular distance of Polaris from the true pole for any hour angle may be found from tables published by the U. S. Geological Survey, and the approximate direction of true north may be computed from a previous determination of azimuth or obtained by a magnetic needle, proper allowance being made for declination.

With the telescope properly focused, set off the computed altitude of the star on the vertical arc and turn the telescope to its estimated bearing. If the star is not seen at once, turning the telescope to the right or left a fraction of a degree without changing the altitude angle will usually bring it into view if the weather is favorable.

For general use the method described by the author for observations on the sun may be simplified somewhat without appreciable loss in accuracy.

In railroad or highway surveys, it is not necessary to observe the sun directly through a colored glass, nor is any prism or other special attachment required, but the sun's image focused on a piece of paper held 2 or 3 in. from the eyepiece of the transit, can be bisected with an error of less than 1 min. of arc. The image of cross-wires shows up plainly when the telescope is in focus.

* Topographic Engr., U. S. Geological Survey, Washington, D. C.

When observations on the sun are made rapidly, the mean of the times and of the angles measured can be used for the computation. The time may be 1 or 2 min. in error without appreciably affecting the results.

Observations on Polaris for azimuth are somewhat more accurate than those taken on the sun, but if they cannot be taken during the day, it is better to use the sun.

D. E. Hughes,* M. Am. Soc. C. E. (by letter).—This paper is an instructive exhibit of the survey methods of the General Land Office, but the methods are not all as scientific as the title implies. On page 545, it is stated that all the temporary corners on a trial meander are moved in the same direction and through distances proportional to their distances from the point of beginning, measured along the meander lines. That statement approximates the probable truth, only as the meander approximates a straight line.

The author assumes that the method is similar to that discussed by Leonard S. Smith, M. Am. Soc. C. E., in 1912,† which is logical, although long, and gives a different result in that the temporary corners are not moved along parallel lines, nor through distances proportioned by the Land Office rule, as given by the author.

A generation ago, a similar problem was presented in *Engineering News* and some other paper simultaneously, and the Profession was surprised to find that few of the many replies were correct. Nearly all followed the rule exhibited by the author, or calculated as is done in balancing a closed traverse. A few used the same method as Mr. Smith, but the prize winner attained the same results by a shorter course, showing that all the temporary corners are to be moved in directions making constant angles with straight lines or diagonals drawn from the point of beginning to them, and through distances proportional to the lengths of those diagonals.

The reasoning, as remembered, was that, in the absence of evidence to the contrary, the field work in both surveys had to be assumed as correct in determinations of deflections and applications of chains, and, therefore, the discrepancy between the point attained by the trial lines and the existing original corner is to be attributed entirely to a difference in orientation, a difference in chain lengths, or a combination of both.

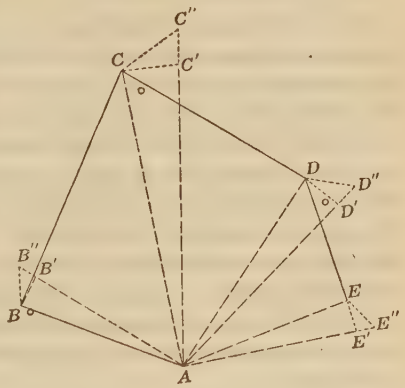


Fig. 13.

Suppose the surveyor assumes a meridian and begins at a known point, *A*, Fig. 13. Following the recorded courses and distances, he runs his trial line through *B*, *C*, *D*, to *E*, near which he finds the original corner, *E''*, and notes its course and distance according to his own survey, from *E*. He then deter-

* Los Angeles, Cal.

† *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 429.

mines by observation, calculation, or plotting, the directions and distances of E and E'' from A, according to his own survey.

If E is not on Line $A\ E''$, his survey must be revolved about A until it is, the rotation being the angle, $E\ A\ E''$. By that process, B, C, D, and E traverse arcs to B', C', D', and E'. The arcs being of equal angular measure, their chords, BB', CC', etc., are proportioned to the radii, AB, AC, etc., and make equal angles with them.

If the point, E', after revolving, is not coincident with E'', his chaining must be altered until it does coincide, the factor to be applied being the ratio of AE'' to AE. By that process, in which all lines are lengthened, or shortened as the case may be, B', C', D', and E' traverse Lines $B'\ B''$, $C'\ C''$, etc., to B'', C'', D'', and E'', these lines being proportional to the radii which they alter.

It has now been observed that, in the triangles, $B\ B'\ B''$, $C\ C'\ C''$, etc., the angles at B', C', etc., are equal, and that the adjacent sides are proportional, each side being to the corresponding side of another as are the radii or diagonals from A to them. The triangles, therefore, are similar. Hence, Lines $B\ B''$, $C\ C''$, etc., along which the temporary corners are to be moved to accomplish the required change in orientation and chain length in one operation, are themselves proportional to the diagonals from A and make equal angles with them.

If the author's rule was applied, the temporary corners would be moved to the points marked by small circles, thus requiring that some lines be revolved to the left and others to the right, and that some be lengthened and the remainder shortened. It hardly seems possible that such erratic procedure, if clearly explained, would be sanctioned by any Court.

W. J. Lightfoot,* Assoc. M. Am. Soc. C. E. (by letter).—The author has shown a wide grasp of a complex subject, and his masterly treatment is not a surprise to those who are familiar with his professional work. Supplementing the statement of improved methods used by cadastral engineers in field work, some of the methods that have been used by the writer may be of interest.

Several direct altitude observations on the sun for azimuth may be taken in a few minutes, but to hold a field party while they are being reduced is a trying experience for one who is endeavoring to accomplish something more than just the taking of observations. One method used to avoid reductions in the field is to take a series of direct observations on the sun each afternoon and reduce them in camp. This makes a reliable check on the results obtained by the use of the solar. If an observation must be reduced in the field, it is often possible to do a large part of the calculation before leaving camp, as follows:

The time and place of starting work is generally known, and the latitude of the place and declination of the sun may be obtained, therefore, in the formula,

$$\cos A = \frac{\sin \delta}{\cos \phi \cos h} - \tan \phi \tan h$$

* U. S. Cadastral Engr., Washington, D. C.

the logarithmic functions can be arranged for the entire reduction, except the cos h and tan h, which may be done quickly in the field after taking the observation.

A plat of the lines to be surveyed on a certain day may be prepared, showing each minute of latitude with its log cos and log tan, as well as a table of the declinations of the sun for each hour with log sin. When the data are arranged in this manner, it is easy to interpolate for a fractional part of a mile or for a fractional part of an hour. After an observation is taken, the only additional reference to the tables is for cos h and tan h.

For all practical purposes, it may be assumed that the ratio of the differences of the vertical angles to the differences of the horizontal angles is constant for one set of observations when the time elapsing between the taking of the first and the last observation is only a few minutes. When several observations are taken, before selecting those to be reduced, find the differences in vertical angles and the differences in horizontal angles. Then, to obtain the ratio, divide one by the other and select those for reduction that agree to the second or third decimal place. In this way, an erroneous observation may be eliminated and not used to vitiate the entire set, as is the case when the mean of a set is used in the reduction.

When the sun, moon, planet, or star is due east or west of the observer, it is said to be on the prime vertical. The altitude and hour angle may be obtained by the following formula:

Let

P = hour-angle.
ϕ = latitude.
δ = declination.
H = altitude.

Then,

$$\cos P = \cos \phi \tan \delta \text{ and } \sin H = \frac{\sin \delta}{\sin \phi}$$

The sun comes to prime vertical in northern latitudes only when its declination is north; therefore, the method can be used for solar observations only for a part of each year.

With accurate local mean time, it would be possible to use a tabulation of the azimuth of the sun made for each 10 or 15 min. of time.

CLAY TALLMAN,* ESQ. (by letter).—The purely technical side of this subject is not in the writer's line; however, during the eight years that he was Commissioner of the General Land Office, he gave much attention to its legal and practical aspects, from the standpoint of results and Governmental policy. As shown by the author, it was largely during this period that the practical experience of previous years became crystallized into settled and established principles and practice. The correct position of the "line fence" has always been a prolific source of controversy and litigation. When it is considered that, as a result of error, obliteration, or fraud, a condition of uncertainty as to property lines often affects large areas, frequently of great value, and,

* Denver, Colo.

further, that in the public land States the survey lines usually constitute the boundaries of political sub-divisions, the necessity and importance of reducing the re-establishment of lines and corners to a scientific system, legal and practical in its operation, becomes at once apparent; this, the General Land Office has worked out, not in theory only, but as applied to literally thousands of actual cases in the field, presenting every conceivable complication. It should not be concluded, however, from this that any engineer with a good general training and a "Manual" is competent to execute acceptable re-surveys; probably in no branch of engineering work are practical experience and good judgment more necessary.

The writer has no criticism to offer of the paper other than to point out that the chart (Fig. 1), on page 520, and the text relating thereto, rather overstates the importance and position of the Division of Surveys, as compared with the field organization, the two branches being co-ordinate under the Commissioner. He would like, however, to emphasize one aspect of the subject not generally appreciated, namely, that in official Government land surveys there exists a unique combination of legal and engineering functions hardly ever found in connection with any other engineering activity. If correctly and legally made and approved, an official survey fixes and determines legal rights; otherwise, the operation fails of its purpose. The first essential of such a survey is lawful authority. The order and instructions for such a survey must emanate from the proper source; the work must be performed pursuant to law by officers qualified legally as well as professionally; and the completed work likewise must receive the approval of the proper officers of the Government. The legal and engineering features are thus inter-related and inter-dependent; one without the other renders the survey ineffective, being in the contemplation of the law no survey. When it comes to re-surveys, this matter of legality of action taken and methods pursued becomes doubly important, because of the inherent complications and because the work must stand the scrutiny of the Department and, in litigated cases, of the Courts. It follows that the closest correlation of the legal and engineering branches of the work is necessary for practical operations; this is accomplished, and has been for more than a century, through the field and office organizations of the General Land Office, pursuant to the laws of Congress.

The consideration of the legal aspect of official surveys suggests a further observation deemed to be pertinent at this time. Much is being said about the re-organization of Government departments and bureaus, with a view to avoiding duplication and correlating activities; among other things, it is urged in certain quarters, that all surveying and map-making activities of the Government be consolidated in a single bureau, and to this end, it is proposed to separate the land-surveying branch from the General Land Office and place it in the proposed "engineering" bureau. From the foregoing, it is apparent that such a course would result in the destruction of a perfectly logical and necessary correlation of activities, in order to create one in name only. Although analysis of other factors affecting the matter, such as the primary purpose of land surveys as distinguished from other surveys, the

percentage of the completion of the land surveys of the country as compared with others, the bearing and importance of the records of all former surveys, and economy and efficiency of operation, furnishes ample reasons why the proposed change should be regarded unfavorably, an adequate conception of the fundamental character of official land surveys, such as the writer has endeavored to point out, should in itself be sufficient. Whatever justification there may be for a re-allocation of other Government activities, the entire matter of land surveys should remain a part of the General Land Office, where it has always been and where it belongs. The only material change in the existing plan of organization requiring action by Congress, which, in the writer's opinion, would be conducive to efficiency and economy in land-surveying operations, would be (*a*) to abolish the offices of Surveyors General and consolidate the work of those offices in the General Land Office at Washington; and (*b*) to provide for one Supervisor of Surveys who, under the direction of the Commissioner of the General Land Office, would have entire supervision of all land-surveying activities, both in the Washington office and in the field.

S. V. Proudfit,* Esq. (by letter).—The writer will discuss this paper from the viewpoint of the public land lawyer, who is interested primarily in the fact of surveys rather than in the means of their accomplishment. This is especially true in the present instance, because of the broad treatment of the subject by the author, in which the limitations imposed by applied science and the law of the land are equally recognized.

The assignment of a definite description by which the locus of a tract may be accurately known is an essential prerequisite to its disposition by the United States; but, until 1910, no means to this end had been devised, either by the Legislative or Executive Branches of the Government, better than the contract system, under which the lowest bidder undertook the survey of a specific area in accordance with instructions he might receive from the Commissioner of the General Land Office. As far as any development of high engineering talent in the Department is concerned, none could have been looked for under the old contract system, and practically none resulted. The employment of the best skill possible in charge of the surveying unit in the General Land Office could not reach beyond matters of organization, for the execution of the work ultimately went to the successful bidder for the contract, whose first interest was in the quantity of his work rather than in its quality. The accuracy of the work performed under such conditions, subjected to all the tests that could be used under the ever-increasing demand for public lands, was, at the best, not susceptible of a reasonable demonstration. This is not to say that the public land surveys during that period were not usually well made, only that they were not as well made as they would have been in the hands of a National body of engineers.

The adoption of the direct system at once provided a permanent foundation for the assemblage of a body of trained cadastral engineers, to whom could be entrusted the duty of executing the public surveys with the cer-

* Member, Board of Law Review, Washington, D. C.

tainty that the latest methods of engineering science would be at the command of the Government. If difficulty had been experienced in securing satisfactory original surveys under the old system, it goes without saying that such difficulties were many times magnified when serious problems of re-survey were encountered. The necessity of protecting vested rights, as well as possible equities, in the work of restoration or substitution, called for the exercise of field control that was practically out of the question under the contract system; therefore, while it prevailed, legislation authorizing resurveys was limited, in effect, to cases where the interests of the Government were dominant. With the installation of the direct system of surveys, however, and the effective demonstration that the public service from thenceforward offered a new field to the trained surveyor, the value of such a body of Government experts received recognition at the hands of Congress in the Act of September 21st, 1918 (40 Stat. 965).

Under prior legislation, resurveys, for the most part, were only made in order properly to mark the boundaries of the public lands remaining undisposed of; but this Act is for the express purpose of furnishing a competent means for the re-establishment of boundaries of land the title to which had passed from the United States.

The first fruits, then, of the direct system of surveys was certainty in result, at less cost to the Government in the matter of original surveys, while the later result was the recognition of a disinterested scientific body of engineers to whom could be referred boundary disputes, by the Courts or individual land owners, and thus reduce to the minimum the consequences of unskillful or reckless field work that had received approval under the old system.

With the adoption of the present system of surveys, it became apparent that a new "Manual of Surveys" should be provided for the Service, and to this end a special committee organized in the Surveying Service of the General Land Office, made a careful study of the situation, its necessities as then existing, and the requisite provision for its future development. Practically, however, the relation of the "Manual" to the public surveys, the working instructions of the Commissioner of the General Land Office, who is charged with all executive duties appertaining to the survey of the public lands, is the same under the new system as under the old, and during the earlier period of transition, changes in procedure were covered by special instructions, and it was not until June 16th, 1919, that the first chapters of the new "Manual" were issued.

This apparent delay in the issuance of this new "Manual", aside from giving the new system an opportunity to develop special needs peculiar to itself, was also due to pending legislation with respect to resurveys, which finally took form in the Act of 1918.

Although all the features of the new "Manual" were given careful consideration from every viewpoint, technical, legal, and practical, when Chapter 6 was taken up, the new field of operations contemplated by the Act of 1918 called for special care in its preparation, and the final review of its provisions prior to adoption will long remain a memorable occasion to the participants in the discussion. With the Commissioner of the General Land Office presiding,

an officer familiar with the public land laws from every angle, the special committee of cadastral engineers, and three members of the Board of Law Review, each with not less than twenty-five years' experience in the adjudication of Federal land questions, the tentative contents of the chapter were considered and discussed line by line through repeated sessions of many hours' duration, with the intent that not only should the public service receive the benefit of the last word in the science of land surveys, but that the vested interests and equities of land owners should be duly protected under the law as construed by the Courts of last resort.

It is a curious comment on the story of public lands and their disposal, that it was not until the greater part of them had been surveyed, and passed out of Federal ownership, that provision was made by which the officer charged with their survey was really so equipped with statutory authority as to insure accuracy in the work. This page, however, of the story is beyond recall, and well it is, for the protection of those who have taken title under the old system, that the protection of their boundaries is not left to the tender mercies of the contractor.

H. B. Patten,* M. Am. Soc. C. E. (by letter).—The resurvey of the public lands of the United States presents an interesting development. It is fortunate that the author has seen fit to call the attention of the Engineering Profession at large to the results of the evolution exhibited in the work of the cadastral engineer engaged in the resurveys.

As in all other branches of science, that of cadastral engineering, when applied to resurveys, has been a gradual and extended development. The first efforts appear to be crude in light of present-day practice. A great advance was made in the public survey when the Government changed from the contract system to the direct method of executing the work. The wisdom of the change has been fully justified in the results obtained in a period of more than eleven years. One of the advantages was in providing the opportunity for more deliberate effort in field work, notwithstanding the increased expense.

It may be of interest to the Profession to call attention to the general progress of resurveys in Wyoming. Among the earliest attempts at resurveys were those authorized in 1891, under Contract 238, in the Valley of the North Platte River in what is now Converse County. Some of the cardinal principles of the resurvey were overlooked at that time, and the work led to so much confusion that it was discontinued. Recently, these lands have been resurveyed, however, in conformity with the most up-to-date practice.

Later, in 1899, resurveys were again undertaken in Wyoming of lands along Fontenelle Creek, in Lincoln County. This work conformed to the essential principles necessary for the preservation of the rights of the settlers, and fairly acceptable results were obtained.

Owing to the great extent of inexcusable error and fraud in the original surveys of the early Eighties, Congress by Act approved May 29th, 1908, authorized the resurvey of 371 townships, nearly 7 800 000 acres of land, in

* Gen. Land Office, Washington, D. C.

various parts of the State. Thus far, only a part of this work has been accomplished.

It has been suggested at times that topographic features should be determined in connection with cadastral surveys and that only one party in the field should be charged with the execution of all this work at the same time; but the fact that the requirements of the two types of surveys vary in many particulars seems to be overlooked by some, so that to carry both systems at the same time clearly increases the expense and delays the ultimate accomplishment.

A radical difference which is not always appreciated, exists between the field and office work necessary to depict the physical characteristics of the country and the intricate work involved in the proper establishment of the division and subdivision lines of the public survey called for in the rectangular system, especially in the re-establishment, restoration, and fixation of section and property lines dependent on the former position of lost or obliterated corners.

In the latter case, there is desired the correct location of the legal position of the section corners and lines as a basis for the proper adjudication of conflicting claims.

J. F. Le Baron,* M. Am. Soc. C. E. (by letter).—It is refreshing to know, even at this late date, that the surveys of the public lands are now being made by real surveyors, and are really made and not "fudged", or scamped. Every one, particularly the people in the West, know the shameful history of these surveys.

The basic plan of the surveys was conceived in wisdom and sound technique, but the method of administration was a disgrace to the nation, and has resulted in untold confusion and needless litigation and has foisted on an unsuspecting and innocent public about 6 000 000 homesteads with insecure and faulty titles, and, too often, no titles whatever, owing to erroneous delineation of boundaries and, consequently, erroneous areas of the sections and subdivisions into which the land was divided.

This era of mal-administration is past, and the members of the Engineering Profession can obtain some comfort from the thought that, by devoting themselves to the rectification of these errors, they can do much to assist the unfortunate victims of a pernicious system.

About forty years ago, while building a railroad in Florida, the writer was in a locality where land had rapidly appreciated in value, and where the public land surveys, which had been made about thirty or forty years before, were in the usual condition of uncertainty and inaccuracy. The writer spent some of his spare time to trying to re-locate last corners and to decide the location of many disputed lines between valuable orange groves. He was appointed a U. S. Deputy and also U. S. Mineral Surveyor to survey some military reservations and phosphate mining claims.

Later, he removed to Jacksonville, Fla., and was engaged to establish the lines of the five old Spanish grants on which that city is built. About 50%

* Cons. Engr., Essex, Mass.

of these grants, at that time, were covered with commercial and residential buildings. The corners of all these grants had long ago been destroyed by the growth of the city, but all the land titles in the city were based on these grant lines and corners. It was no easy task, but after nearly a year's work, they were re-located satisfactorily.

In recovering several of the old corners outside of the compactly built up city (Jacksonville), the tap-roots of the hard pine witness trees were found. The writer has often found old well blazed section lines leading to a point 20 to 75 ft. from the established corner, which corner might be also identified by other blazed lines leading to it from other directions and by well marked and identified witness trees. Generally speaking, the distances from the corner to the witness trees were not measured, but were either paced or estimated, although the courses given in the original field notes were generally correct. Lengths of section lines generally overrun 10 to 30 links, making the areas of the sections larger than the acreage given on the official plats. It would be interesting to know how many acres of land the Government has lost by this error in the whole of the public lands. It is safe to say it would be up in the millions of acres.

The fraudulent surveying of shore lines of rivers and other bodies of water has worked serious harm. Intending settlers or purchasers, deceived by the officially certified plats furnished by the Government, have selected lands, paid for them, and often settled on them, only to discover the shore line mislocated and possibly a bay or other body of water where the Government plats showed high land, and their acreage thus reduced, oftentimes, from 10 to 20 acres.

In 1876, the surveys had been completed in ten States and Territories, and were in progress in twenty others. They are now practically completed, except in Alaska and Utah, and resurveys in some States. Omitting Alaska, the arable acreage of which is unknown, there remains 19 000 000 acres to be surveyed in Utah.

Howard Richards Farnsworth,* Assoc. M. Am. Soc. C. E. (by letter).—The character of the subject chosen, together with the definite intention on the part of the writer to present to the Profession a résumé of those principles which official experience has proven will, when properly applied, assist the engineer in the solution of any problem of resurvey encountered, whatever the degree of complexity, has precluded any departure into the field of original Government surveys.

It may be stated, however, that the instrumental equipment and the field methods used for the projection and measurement of lines of both surveys and resurveys are much the same, but the problems encountered in the former, although difficult, are usually of a distinctly different nature—the element of private interest and vested rights not having such primary bearing. This reference, of course, does not include those original surveys executed of lands sometimes found along meanderable bodies of water, of islands, fictitious lakes, or other tracts, which it develops were erroneously omitted from the original

* U. S. Cadastral Engr., Gen. Land Office, Div. of Surveys, Washington, D. C.

township survey, nor does it include those other surveys, which the General Land Office alone is authorized and qualified under the law to execute, required for the identification of lands for purposes of evidence in suits or proceedings in behalf of the United States. This class of fragmentary work involves questions of status and classification as to survey and, thus, becomes closely allied to resurveys, inasmuch as the private rights involved are generally of paramount importance. The investigations and surveys which are now being executed in the bed and flood-plain of the Red River in connection with the Oklahoma-Texas boundary litigation, form an example of work of this class.

Mr. Douglas properly emphasizes the value of a correct reference of the basic system of the lines of any public works to the true meridian, as incidental to a finished return. In any long traverse such as a railroad, canal, or highway survey, where there is often no opportunity for closure, correct bearings of lines are especially desirable. In such cases, the reported definite relation of each line to the meridian will provide the means by which to identify each line of the survey although only one point of the whole system may be known.

Attention has been called by Mr. Douglas to the ease with which the necessary co-ordinates of Polaris may be found in tabular form, for use in the identification of the star during the daylight hours. It might also be well to recall that precise tables of this character may be found in the "Nautical Almanac" published by the Naval Observatory.

A similar tabulation also appears in the 1921 edition of the "Ephemeris of the Sun and Polaris," to which pamphlet frequent reference was made by the writer in his paper. This table was formulated by the General Land Office to facilitate the determination of the latitude by an altitude observation of Polaris at any hour angle, the latitude of the station being determined by applying a primary and supplemental correction to the observed altitude of Polaris, properly corrected for refraction. The tabulation is concise and practical in its operation and the vertical angular values of Polaris above or below the pole for any mean time hour angle are readily taken from it. From a station of known latitude, the position in altitude of Polaris is thus easily determined. The position in azimuth of Polaris for a known latitude and a stated mean time hour angle and declination is readily apparent by reference to the table of azimuths of Polaris also to be found in this "Ephemeris". By the use of these two tables, the position of Polaris is easily found for any hour angle and latitude, from any station within the United States.

Mr. Hughes exhibits much assurance in the revival of an old subject. The exposition of his method of traverse adjustment is made to the exclusion of all others, and it is championed with the courage of true conviction.

The earliest presentation of this "twist" method as far as the writer knows was made by Mr. F. Hodgeman,* in 1895. Mr. Hodgeman, however, guards the method properly by restricting its application to cases where it might definitely be known that the angular measurement and rhumb-line projections were perfectly accomplished and, therefore, that the whole error might

* "Manual of Land Surveying", pp. 78–81.

theoretically be charged to defective measurement and the initial meridian determination. He explains,* however, that in practice numerous additional errors occur from many causes and he then reiterates the first fundamental principle of balancing, a rule which has not ceased to be as sound in logic as it is unassailable, as follows:

"As the sum of the lengths of all the courses is to the length of each course, so is the total error to the error of that course."

The method of traverse adjustment so vigorously championed by Mr. Hughes was known, discussed, and its limitations fully recognized many years ago by the Cadastral Engineering Service and has since been considered to be inapplicable to the reproduction of meander lines. In all other cases where this "twist" method has been brought to the writer's attention, its advocates have claimed for it only the limited application possible under a positive assumption as to the specific manner in which the errors in the survey were made, rather than the negation appearing in the expression, "in the absence of evidence to the contrary". As Mr. Hughes makes no attempt to confirm this negation, it follows that if there is no evidence in opposition to the specific assumption then there is also, by implication, certainly none in its favor, and the amazing conclusion is then reached that, in the absence of any knowledge whatever as to the contributory causes, all the error for this reason must be definitely assigned to the performance of certain functions and excluded from others.

According to the most elementary conception of the law of probability, if a lack of knowledge or evidence is admitted as to certain causes (the functions performed by instrument and chain), all elements contained therein must share responsibility in the known, combined effect (the closing error found). In the absence, therefore, of definite knowledge to the contrary, both instrument and chain in performing all their functions must be held to be mutually responsible for errors found, since each, independent of the other, measures one of the two polar co-ordinates necessary for the fixation of a definite point on the earth's surface. The combined effect of the instrumental errors in azimuth, angular deflection, and adjustment, are chargeable with the same degree of inaccuracy as the combined effect of chain errors in leveling (or reduction to horizontal by clinometer), in the alignment by chainmen, and unit errors in the length of chain.

It should not be necessary to recall the fundamental principle that, when two or more independent measuring functions are performed to determine the co-ordinates of a point, the degree of accuracy with which that point is fixed substantially depends on the accuracy obtained by the least accurate of the various functions entering into the process. To perform the linear measurement in a definitely and materially less accurate manner than the angular measurement and projection of lines, would involve a loose and rash expenditure of time in the accomplishment of an accuracy which serves no purpose. A surveyor will avoid any ineffective refinements and, unconsciously, will regulate the operation of instrument and chain so that if either has a limit of accuracy materially different from the other, a needless waste of time will

* "Manual of Land Surveying", pp. 120–121.

be avoided by no attempt at refinements materially inside the limits of the least accurate. This is certainly true as to the later survey or the resurvey which develops the error and, without doubt, is also true of the original survey. Naturally, then, a bold assumption as to the specific cause of error, that the discrepancy between the results of the original survey and the resurvey can "be attributed entirely to a difference in orientation, a difference in chain lengths, or a combination of both," and that, therefore, the error found was due entirely to erroneous orientation and chain lengths in both surveys, is particularly baseless and untenable.

Although it is recognized that the mechanical errors in chaining are compensative and subject to one law and that the unit errors in chain length and errors in alignment are cumulative and subject to a different law, the only permissible hypothesis as to the combined error of both chain and instrument is, that this combined error, which in its combined effect represents the result of all contributory causes, varies directly with the length of the line surveyed. When this conclusion is reached, the method of restoration follows as a matter of mathematical certainty and is that given in the paper. Any other hypothesis makes necessary an assumption of a definite knowledge as to the particular contributory causes which were previously held to be unknown. The question, then, is primarily one of hypothesis, for the proper method will follow as a secondary matter of accomplishment. For instance, in those cases where it is definitely known that the original surveyor inadvertently has confined all the error to the measurement and the initial azimuth, or perhaps, with a view to the ultimate application of this "twist" plan of restoration, has purposely done so, then, without doubt, the method set forth by Mr. Hughes follows with equal certainty.

In the case of a meander line, the angle-points of the traverse are established without monuments, except at the initial and terminal points, and the known error is that which results from the extension of the record bearings and lengths by a later and presumably more accurate survey from either existing terminal monument toward the other, or from both toward the particular angle-point to be recovered (see Figs. 9 and 10). The direction and extent of the error thus developed is the same for all points along the traverse; that is, it must be considered from the record of a single later survey, regardless of its order of recorded progress, that it was extended from either existing terminal monument toward the other or from both toward any one of the angle-points, on the original record bearings and lengths; in which case, the error developed will be the same for all points.

Considering any one of the angle-points for the purpose of illustration, the recovery of its original position by a later survey must depend on and be influenced by all the existing evidence of such original position. The sum total of all this evidence is the two existing terminal monuments and the record of the original survey as to the courses and distances therefrom. The only logical manner whereby the influence of the known evidence can be properly exerted on the ultimate recovery of the point in question, is by extending from each existing monument the original record courses and

distances until the angle-point is reached where, unless no error is developed, two separate points for the position thereof will be found (referred to hereinafter as temporary points, see Fig. 9). If only one original terminal monument is assumed to be known, then the position reached for the angle-point from this monument should be adopted at once as the ultimate position thereof. Similarly, if the other of the terminal monuments is known, and the first one is lost, then the other of the two temporary points would be adopted with equal assurance as the final position. However, in the presence of both terminal monuments and two separate temporary points, the true position for the restored angle-point must be such that it will be nearest to both the temporary points indicated by the terminal monuments, or somewhere on a straight line between them. If the errors vary directly with the length of the line measured, this restored point will occupy such a position on this straight line, that it will divide the line into distances which are proportional to the length of the lines measured from each terminal monument.

By presenting the problem in this manner, the careful reader must see at once that each existing original monument and all the calls of the original record therefrom have been made to exert their proper influence in the recovery of the required angle-point, and he will realize that not only all legal requirements are fulfilled thereby, but the scientific requirements as well. The theory of non-concurrent parallel forces might be applied to this problem, or the principles involved in a determination of the "center of gravity", but the same resultant point would be reached. No one element whether it is the known monuments, the bearings, or the distances, can be regarded as evidence less competent than any other and to neglect any one of them arbitrarily and without reason, would be indefensible.

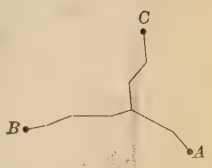


FIG. 14.

Consider, now, that, in the original record, a third existing corner monument had been connected by a traverse line to any one of the angle-points of the first traverse line, as shown in Fig. 14.

It would be interesting if Mr. Hughes would apply his method to the restoration of the angle-point common to both traverse lines. If each of the three existing corners was to share in the control of the restored angle-point, he would have no option, but to apply his method to the three pairs of corners separately, that is, first between Monuments *A* and *B*, second between Monuments *A* and *C*, and, third, between Monuments *B* and *C*. In this event, three different positions for the restored angle-point would result, and he could not escape the necessity for consideration of a mean position of the three for his final position. A logical extension of this plan of restoration would also give two separate positions for every angle-point, other than the common point just referred to, in the tri-system. By determining a mean position for all such points and connecting them with right lines, what then would become of the proportional lengths of lines and the constant angle of twist?

In contrast with the inconsistency produced by the foregoing, the application of a method will be investigated, conceived in the assumption that all

existing corners and all the calls of the original record are to share alike in a proportioned influence on the final position of the angle-point to be recovered.

Extend the lines on the record bearings and lengths from each existing monument (*A*, *B* and *C*), to each angle-point in the tri-system. This gives at once three temporary points for each angle-point of the system. Each of these points being weighed inversely as its distance from the corresponding existing monument, the mean position is found by applying the principle of non-concurrent parallel forces as explained in the paper (see Figs. 8 and 9). The application of this method is general, no matter how many existing monuments are considered as being connected with the point, the restored position of which is required, and no matter how complicated the system of the record survey may be. The results, in all cases, are logical and consistent.

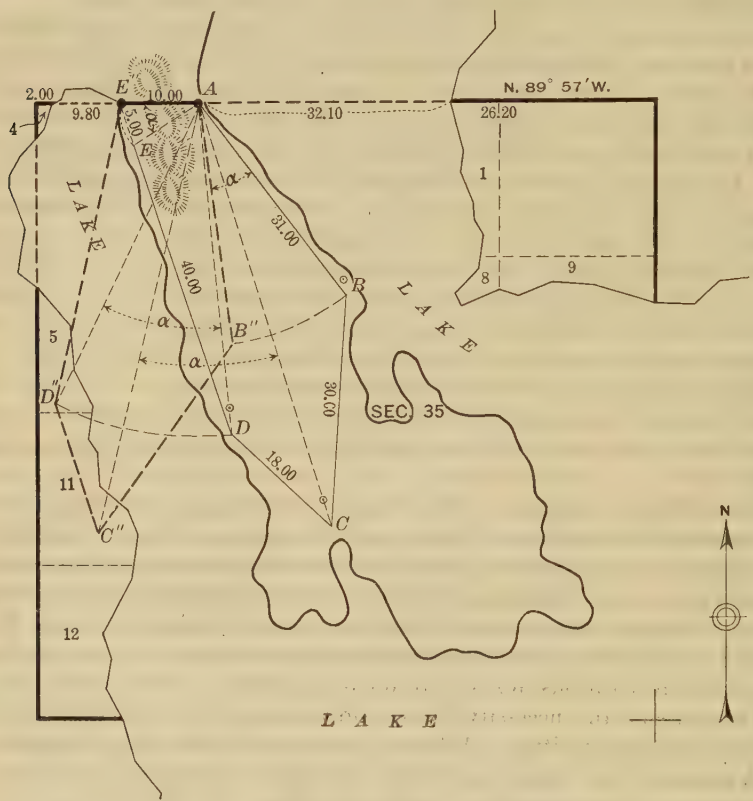


FIG. 15.

The absurdities which must be expected in any general application of the Hughes plan of restoration are only partly evident from Fig. 13. To make it more apparent, attention is invited to Fig. 15, which illustrates the restoration of the erroneous original meanders of a peninsula. This arrangement of the lines of the traverse is not unusual, and is often en-

countered also in the fixation of the boundaries of fictitious lakes. It will be observed that the assumed closing error is not excessive and may well be expected in view of the relatively long distance traversed. All the angle-points and terminal monuments have been designated in this diagram with letters corresponding to those assigned by Mr. Hughes in Fig. 13, in order that the restoration can be easily accomplished, following the plan as explained by him. A fanciful position will immediately be occupied by the traverse system after its rigid orientation in azimuth, in fact, such resultant positions will sometimes reach into entirely different quadrants. The corresponding positions occupied by the angle-points, when reproduced by the application of the only logical method, are those indicated by the small circles.

It is believed that Mr. Hughes has either not read the paper as carefully as he should or else that he has allowed his judgment to be influenced more by the apparent geometrical elegance of the method he advocates than by a proper consideration of what is to be accomplished.

Mr. Tallman furnishes conclusive evidence of the keen interest that he took in the work of the Cadastral Engineering Service throughout his incumbency as Commissioner of the General Land Office, and of his intimate knowledge of the general subject from the executive and legal point of view. As he has stated, it was largely during this period that previous experience developed into established principles and practice. He realized that, to a large extent, the adjudications of the Land Department and the operation of the various land laws must rest on and be influenced by the preliminary determinations of the cadastral engineer.

Effective safeguards for the elimination of errors in field observations, and practical suggestions for saving time in the resulting reduction operations, are frequently developed and verified through individual use. Such, apparently, is the case with Mr. Lightfoot, who has made valuable suggestions of methods he has used. Where one finds real efficiency in the expenditure of time, with a due regard for accuracy, one will also find the origin of progressive ideas, and investigations looking to the development of new forms. Mr. Lightfoot's suggestion of the possible use of a tabulation of azimuths of the sun is interesting, and the writer would like to have had him pursue the investigation of such a tabulation further, in order to determine its practicality.

There is no one who deserves more studious attention in a discussion on the fact of cadastral surveys, their foundation in law, and their legislative development than Judge Proudfit, who has given a most interesting discussion of the subjects on which he has touched. Particular attention has been given by him to the advantages immediately accruing to the Government in the abolition of the contract and the establishment of the direct method of execution. Mr. Patten and Mr. Le Baron have also stressed this point. When it is considered that the public domain is the people's most valuable heritage, it seems that the principle of conservation, as exemplified by proper facilities for accurate identification of the units of disposal, was applied to it none too soon. The Government's industrial experiences

during the World War have furnished renewed and costly examples of what the public may continue to expect in its relations with an unscrupulous contractor. "Certainty in result" in this period of enlightenment surely should continue to be the primary object. The maintenance of a National body of civil engineers within the organization of the General Land Office to administer and execute the complicated work of Federal cadastral surveys and resurveys, as so aptly referred to by Judge Proudfit, is a natural pre-requisite to the attainment of certainty in results. The writer knows of few instances involving scientific endeavor, where the Government is obtaining satisfactory results through the medium of a contractor, even in the presence of the most rigorous safeguards.

It should be stated, in this connection, that those efforts toward perfection in results that this body of engineers is consistently maintaining, have been materially aided by the confidence and constructive co-operation of the members of the Board of Law Review of the General Land Office. The Cadastral Engineering Service is fortified in the knowledge that the conclusions and material prepared by its Manual Board for the first six chapters of the new "Manual of Surveying Instructions", stood with steady permanence the searching inquiry and critical judicial consideration of three such authorities on the jurisdiction and practice of the United States Land Department as Judges Samuel V. Proudfit, John McPhaul, and William H. Lewis, who participated in the closing conferences on these chapters before official approval. The familiar tradition involving the supposed indifference of the lawyer toward the engineer, certainly did not apply in the closing conference on the "Manual", recalled by Judge Proudfit, when he and his colleagues; the Commissioner; Mr. Frank M. Johnson, Supervisor of Surveys; and the Manual Board composed of Mr. Walter T. Paine, Chairman, associated with Messrs. Arthur D. Kidder, Thomas C. Havell, William H. Craigue, and Clinton G. Tudor, reached a unanimous agreement on the material prepared and presented by the Board for final consideration. When it is recalled that this work had previously received the endorsement of Mr. Charles L. Du Bois, for twenty-five years Chief of the Division of Surveys and Resurveys, its firm foundation in science and law will be duly appreciated, commensurate with the extent and quality of all the effort expended in its production.

Mr. Patten has reviewed certain interesting events in connection with some of the resurvey projects in the State of Wyoming, and Mr. Le Baron recalls the difficulties he encountered in the recovery of the obliterated evidence of the early surveys in Florida. His experience with the restoration of the lines of the five old Spanish land grants on which the City of Jacksonville is based, is illustrative of the difficulties encountered in the restoration of the irregular boundaries of some of these old grant lands, which are now often of great value. A recent example of a similar resurvey is the reproduction by the Cadastral Engineering Service of the boundaries of a tract, known as Rancho Potrero Chico, in Ts. 1 and 2 S., R. 11 W., S. B. M., California, in the suburbs of Los Angeles. In this case, clear identification of the boundaries of the granted lands had not been provided until the filing of the resurvey plat in 1920. As clear title had never been asserted to this tract,

a United States patent covering it had still to issue on that date. The several claimants of record and parties in interest included eight individuals, the State of California, and one oil corporation. The tract was reported to be worth $1 000 per acre for agriculture and, in addition, to be covered with producing oil wells, thus making the property exceedingly valuable. Mr. Lightfoot will recall this interesting case, as he participated in the field work. The old archives of Spanish and Mexican origin, which are to be found in the files of the local offices of U. S. Surveyors General, include numerous interesting documents, many of which bear the signatures of the reigning monarchs and the rulers of those countries.

Mr. Le Baron's implication that an irrelation generally exists between the legal acreage as shown on official plats and actual conditions on the ground, and that there is always a corresponding loss of land by the Government, is somewhat misleading. As a matter of fact, it is found that the ordinary theories of probability are fully verified by the results of boundary retracements, other than those of limited scope, and that excesses and deficiencies in original measurements are developed with equal frequency. It might also be well to correct the statistical data which have been given as to the extent of completion of the subdivision of the public domain, and the present status of resurveys. The public land States include all the States, except the thirteen original States and Maine, Vermont, Kentucky, Tennessee, West Virginia, and Texas. The total land surface of the public land States is estimated at 1 442 200 320 acres, or more than three-fourths of the total area of the forty-eight States. The Territory of Alaska is estimated to cover 378 165 760 acres. On June 30th, 1921, about 168 000 000 acres of public land in the public land States had not yet been surveyed. In the Territory of Alaska, the area which has been surveyed is insignificant compared with the total area of the Territory. Retracements and resurveys have been and are now being executed in nearly all the Middle and Far Western States and in many Eastern-Southern States. As settlement and land values increase, the demand for Federal resurveys increases and often exceeds the available facilities for supply.

Land patents do not convey title primarily to a stipulated number of acres, but rather to certain designated units of disposal as fixed by the boundaries thereof. The area content is secondary and is that which is actually found to be embraced within the stated boundaries. Consequently, the writer is somewhat at a loss to understand what was intended by Mr. Le Baron in the suggestion that land patents had sometimes been issued which conveyed "no titles whatever." The nearest approach to this condition might be in a case of partial or total conflict of one claim with another, a situation which ordinarily must have been known to the affected entrymen, but had not been revealed to the General Land Office until the patents had been issued and were beyond recall. In such a case, the proprietary authority conveyed by the respective patents would be properly commensurate with the good faith of the entrymen during the progress of the entries toward the conferring of the titles, as each conflicting claimant would be fully aware of the equitable and relative rights of the other. If a just adjudication was made, neither claimant could rightfully consider himself to be injured.

The idea that patents might sometimes not convey title, even though not confronted by any adverse or superior claim, reminds one of the suggestion sometimes advanced that when the record of the original survey, in a given locality, cannot be reconciled with the relative positions of the existing corners, or that when no physical evidence whatever of original monuments can be found, the inference is justified that the original surveyor did not run the lines as he stated in his field notes and, therefore, could not have built any corners; in which event, it is then maintained that the original corners cannot be recovered in a resurvey, or restored for the control of the boundaries of alienated lands.

Such a conclusion is based on entirely false premises. Resurveys do not attempt to reproduce that which once existed and is lost, but, rather, they reproduce that which the original record purports to exist. The physical corner monument itself is, of course, the best existing evidence of the position of the original survey as described in the record; however, if all the monuments had been originally established and were afterward lost, the same problem would confront one, as in the case where they were never established. In either case, one has the positions where the original record claims they were, and, therefore, it is not vital to the restoration whether or not the physical monuments were ever actually established, since, in both cases, all physical evidence thereof has been considered to be lost. The original survey, when official, is an enduring basis for disposal, and land titles are not to be invalidated at any time through the destruction of the corners. The survey is made enduring through the filing of the official field notes and plats. The proper interpretation of this original record, as referred to local conditions, for determining questions of good faith in location, makes necessary the application of the definition of good faith, such as is given on page 551.

A claimant is not necessarily involved in litigation in the adjudication of an area in conflict (see Plate V). In general, a conflict is subject to clearance by private mutual agreement, adjudication by a competent Court, or, when either party admits and asserts no proprietary right to such an area and voluntarily relinquishes all interest in the same, then, also, by the law providing for the amendment of entries or selections and re-issuance of patents, Section 2372, R. S., as amended by the Act approved February 24th, 1909 (35 Stat., 645). On a satisfactory showing by either party and reconveyance to the Government of unencumbered title covering the original units of disposal invaded by such conflicts, a re-described entry or patent will be issued by the General Land Office, reaffirming his interest in and title to the land, exclusive of the conflict. In these cases, the claimant is then allowed to take in lieu thereof, any equivalent area of vacant available land of appropriate class, which may be contiguous, or if none is adjoining, then to complete his homestead, selection, or other right, by taking any available equivalent area of vacant land, which may not be adjoining, the latter in the form of a supplementary entry or selection under the entry laws. The particular plan adopted for the clearance of any conflict depends on the attitude assumed by the parties in interest; however, in the end, each claimant should receive that to which he is entitled.

The writer has reviewed in some detail the subject of conflicts, in the belief that the discussion will be of some value to engineers when they encounter these perplexing problems in the field. Concluding the discussion, there appears to be a general agreement that the subject of cadastral resurveys requires the most deliberate consideration by the Engineering Profession, in order that when attempts at restoration are made, the fixation of land boundaries and the delineation of private rights may be accomplished with finality.

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Paper No. 1490

BUCKLING OF ELASTIC STRUCTURES

By H. M. Westergaard,* Esq.†

With Discussion by Messrs. George Paaswell, Eugene E. Halmos, Edward Godfrey, and H. M. Westergaard.

Synopsis

The buckling of a slender column under an axial load and the collapse of a thin cylinder under a uniform external pressure have long been recognized to be related structural phenomena. In both cases, the structural action may be described by the common term, buckling. If the column is perfectly straight, the load on it exactly central, and the material perfectly homogeneous and elastic, when the load is increased gradually from zero, the column remains straight until the load reaches a critical value at which the column bends out suddenly. In the same way, if the cylinder is exactly circular, the outside pressure on it absolutely uniform, and the material perfectly homogeneous and elastic, when the pressure is increased gradually from zero, the cylinder remains circular until the pressure reaches a critical value at which the cylinder deflects from the circular shape. In both cases, the deflections increase at the critical value of the load without any increase of the load. Until the deflections have become large, there is a neutral equilibrium, that is, one maintained at constant load throughout a continuous range of configurations. This neutral equilibrium is the criterion of what may be termed pure elastic buckling.

If the column is not perfectly straight, if the axial load on it is slightly eccentric, or if there are transverse loads in addition to the axial load, and, in the same way, if the cylinder is not perfectly circular and homogeneous, or the load on it not exactly uniform, then certain elements of the pure buckling may still be traced, and the structural action may still be characterized as buckling. The deflections increase slowly at first, with slowly increasing loads, but they increase rapidly as the loads approach the critical values. This type of buckling may be looked on as a mixed action lying between two extremes, one

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of which is the pure elastic buckling first mentioned and the other the ordinary structural action in which the deformations and stresses are proportional to the loads, for example, the ordinary bending action of a beam loaded by transverse forces only.

There is, as may be seen from the Bibliography (Part VII), a rather voluminous literature on buckling. Most of the analyses made in the past deal primarily, however, with the various cases of pure buckling and leave the questions of mixed buckling on the whole unsettled. The present investigation deals primarily with the subject of mixed buckling. A general analysis is made of this type of buckling, and the results are applied to various specific cases.

The method followed, to some extent, is to present results first, and theory afterward. For example, the formulas in Parts II and III are derived from the general formulas in Parts IV and V. The subject-matter is arranged as follows:

I.—Introduction (Articles 1-2), in which certain limiting assumptions are introduced, examples of buckling are discussed, and some terms are defined.

II.—Formulas Applying to Columns Which Are Loaded by Axial and by Transverse Forces at the Same Time (Articles 3-9), in which approximate and exact formulas are given for columns with hinged or fixed ends, loaded by a distributed or by a concentrated transverse load, or by end couples.

III.—Slabs and Similar Structures (Articles 10-11), in which formulas are given for rectangular homogeneous slabs, simply supported on four sides, and for a double system of crossing beams connected at the cross-points.

IV.—General Theory of Structural Actions in Which the Stresses Are not Proportional to the Loads, Although Hooke's Law Applies in Each Element of the Structure, and Although the Deformations Are Small (Articles 12-27), in which by the principle of energy minimum some general formulas are derived, by which one may compute any structural effect, such as bending moment, stress, or deflection, in mixed buckling, provided the corresponding case of pure buckling is known.

V.—Differential Equations Applying to Some Definite Cases (Articles 28-31).

VI.—Summary (Article 32).

VII.—Classified Bibliography.

I.—Introduction

1.—Limiting Assumptions, Examples of Buckling, and Definitions of the Terms Orthostatic, Astatic, and Heterostatic.—Two assumptions are usually made in the analysis of stresses in structures, which have been shown by experience, by test, and by analysis, to be warranted within a wide range of cases, and they will be made throughout this investigation. One assumption is that at all points the unit deformations, or strains, are proportional to or are linear functions of the stresses; the other is that the elastic deflections from the original shape are small and may be considered as infin-

itesimal when compared with the main dimensions of the structure, or, in this sense, the structures are assumed to be stiff. These two assumptions, of elasticity and stiffness, limit and simplify the problem to be investigated.

When both assumptions are warranted, one may conclude in many, although not in all, cases that a stress or a deflection produced by two loads combined is the sum of the stresses or deflections produced by each of the two loads acting separately; that is, the principle of superposition is found to apply to the loads, stresses, and deflections. The general principle of superposition, when it applies, states that the effect of two or more causes combined is the sum of the effects of the separate causes; for example, the stress due to a total load consisting of live load plus dead load is computed, in accordance with this principle, by adding the live load stress to the dead load stress. An important conclusion drawn from the law of superposition of loads, stresses, and deflections is that when all the external loads are increased in a certain proportion, the stresses and the deflections are increased in the same proportion.

It is evident that the law of superposition does not apply generally to the cases of buckling which have been mentioned. In fact, one may define elastic buckling in the most general sense as a structural action in which the stresses are not, in general, proportional to the loads, although Hooke's law applies in each element of the structure, and although the displacements are small; or, as a structural action in which the assumptions of elasticity and stiffness apply, but in which the law of superposition of loads, stresses, and deflections does not apply to the structure as a whole, although it may apply to certain parts of it. It is intended to investigate elastic buckling in this sense of the term.

A special nomenclature for the two actions which have been referred to as pure buckling and mixed buckling will be introduced later. First, the nature of these actions will be explained by means of examples.

Beginning with pure buckling: The example, already referred to, of the straight slender column buckling under a central axial load, shows the most characteristic features. Let the ends be simply supported or hinged, and the cross-section be constant. Then, at the critical load, given by Euler's well-known formula, the transverse deflections which were zero or negligible at smaller loads, increase suddenly. The neutral equilibrium, maintained at this constant critical load throughout a continuous range of increasing deflections, is the criterion of pure buckling. The load may be increased beyond the critical value at which the bending ordinarily begins, and still the column may be made to remain in equilibrium in an undeflected state. This equilibrium is unstable, however; that is, it would be disturbed by a small force from the side, or by a small local eccentricity of the elastic resistances of the cross-section. At higher "critical loads", other states of neutral elastic equilibrium may be obtained, in each of which the column may buckle into a curve consisting of some definite number of half waves. All the neutral equilibria produced under the higher critical loads are unstable. When they do occur, they will tend to revert to the neutral equilibrium which was produced first, and the maximum load which can be applied, will drop to the corresponding lowest critical value. The lowest critical load at which buckling first occurs

is of particular interest, because it defines a limit of the strength, but it will also be shown, subsequently, that the study of the higher critical loads and of their corresponding neutral equilibria leads to a solution of important structural problems.

Fig. 1 illustrates some typical cases of pure buckling of elastic structures. Fig. 1 (*A*) to Fig. 1 (*F*) shows some originally straight columns with various end conditions. Column *A* has hinged ends, Column *B* fixed ends, Columns *C* and *D* have one free end, while Columns *E* and *F* have elastic supports at the ends. In the case of Column *C*, the load at the free end is assumed to remain parallel to itself while the deflections increase, while in Column *D* the load applied at the upper end of the column is inclined in proportion to the deflection of that point. If Columns *C* and *D* are of the same dimensions and material, then Column *D* will buckle at a lower load than Column *C*. In the cases of Columns *E* and *F*, the same types of buckling may occur as in the case of Column *A*, but, in addition, neutral equilibria may be produced by the deformation of the spring supports, whereby the column moves into a position which is inclined with respect to the vertical, as indicated in the diagram. Fig. 1 (*G*) shows a section of a long, thin-shell cylinder under uniform outside pressure. When the pressure reaches a critical value, the surface buckles, as indicated by the dotted line. The typical neutral equilibrium is found here also. Furthermore, a larger number of waves in the deflected surface might be formed at higher "critical pressures". Only the lowest critical load gives a stable neutral equilibrium. All the neutral equilibria which could be maintained at the higher critical pressures, are unstable; that is, the general characteristic features mentioned for a slender, straight column exist also in this case.

Fig. 1 (*H*) shows a curved compression member which carries forces at the ends and, in addition, a distributed pressure from the convex side. If the original center line is a possible line of pressure, that is, if it is a funicular curve for the side pressures, then buckling may take place, as in the case of the circular cylinder. Fig. 1 (*J*) shows a bridge of the pony-truss type; the sidewise buckling of the arch is restrained by the stiffness of the verticals, but this restraint may be overcome at a certain limiting pressure in the arch. Fig 1 (*K*) and Fig. 1 (*L*) represent slabs loaded from the ends; the case shown in Fig. 1 (*L*) occurs, for example, in the web of a plate girder under the influence of the shear. The edges may be either fixed or free to turn. At critical loads, the slabs will buckle into various surfaces. The case shown in Fig. 1 (*M*) is similar to that in Fig. 1 (*K*), only the continuous slab is replaced by a double system of crossing beams. Fig. 1 (*N*) is a hinged-end column with a continuous elastic support from the side; the springs shown in the diagram are assumed to be without stress when the column is straight. The possible curves of deflection are similar to those in Fig. 1 (*A*), but the critical loads, if the column itself is the same, are essentially higher, and the number of half waves of the deflected curve corresponding to the lowest load is likely to be two or more, depending on the stiffness of the side support. Fig. 1 (*O*) shows a cylinder under outside pressure, but with the surface stiffened by rings. The relation of this case to that of the cylinder, Fig. 1 (*G*), is similar to the relation existing between the columns, Fig. 1 (*N*) and Fig. 1 (*A*). If the two cylin-

ders are similar, Fig. 1 (*O*) will collapse at a higher load than Fig. 1 (*G*), and it is likely that a larger number of surface waves will be formed in the process of buckling. Finally, Fig. 1 (*P*) shows a truss column, which may buckle in the same way as the column shown in Fig. 1 (*A*).

Other examples may be mentioned: A long shaft or wire may buckle into a spiral shape under the influence of either torsion alone or torsion combined with an axial load. This action is most easily demonstrated by twisting a short piece of string. A high narrow beam or blade may bend out sidewise, tip, or twist, under the influence of a bending load which lies initially in the plane of maximum rigidity, that is, in the plane of symmetry containing the large dimension of the cross-section. At a certain critical value of the load, the torsional rigidity becomes insufficient, and the beam tips. This action is demonstrated when one tries to hold a long strip of paper or a T-square as a horizontal cantilever with the flat side vertical. A thin cylindrical shell or a tubular strut may buckle into a double-curved surface under the influence of a compressive load parallel to the axis. Buckling may occur in a spherical shell loaded by a uniform outside pressure. A closely related dynamical action is the whirling of a shaft, whereby the shaft bends out when a critical speed has been reached. Concerning the larger number of cases of pure buckling which have been mentioned, reference is made to the literature in the classified Bibliography (Part VII). As far as pure buckling is concerned, the present investigation deals mainly with the general aspects of this action and with its relations to mixed buckling.

The nature of mixed buckling will now be explained. Mixed buckling may occur in any structure in which pure buckling is possible; for example, with any of the structures shown in Fig. 1. A load must be acting which can be resolved into two component parts: One, a load which alone would produce stresses and deflections that are proportional to it in accordance with the principle of superposition; the other, a load which alone would produce pure buckling when increased to a critical intensity. Assume that the two loads produce deformations and stresses in the same parts of the structure. Assume also that the two components of the load act together and are gradually increased in the same proportion from small initial values. Then, a structural action will take place in which, in general, the stresses are not at any stage proportional to the loads, and in which the law of superposition does not apply at any stage except, possibly, to stresses at special points or to special deformations. This action differs from pure buckling. The load is a mixed load, and, therefore, the action may be called, properly, mixed buckling. An example is the column loaded endwise and sidewise at the same time. Evidently, each case of mixed buckling is closely related to a corresponding case of pure buckling.

As no general nomenclature has been developed in technical literature for the actions described, it seems desirable to introduce one. The terms, orthostatic, astatic, and heterostatic are proposed for discussions of these actions, and they will be used in this paper. The first term denotes the action in which the law of superposition applies. As this action may be said to represent the regular case, it will be called the orthostatic action.

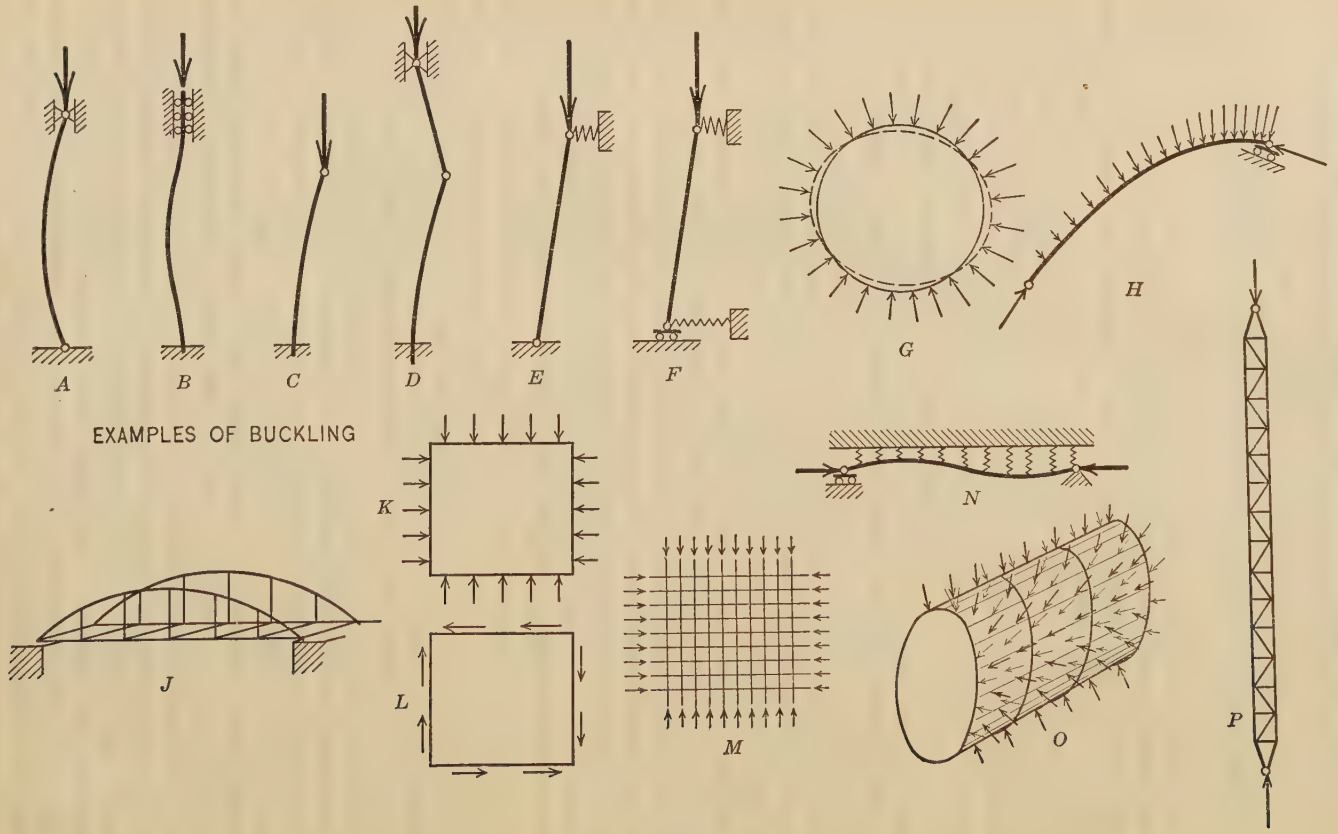


FIG. 1.

The second term will denote the action in which an increase of all loads in the same proportion to critical values produces pure buckling, characterized by a neutral elastic equilibrium. Such action will be called astatic action. This term is explained by the common usage of the word, "astatic", as indicating certain conditions under which a neutral equilibrium is created. The neutral equilibrium is sometimes called an astatic equilibrium, a usage which will be followed by the writer. With the terms introduced, it may be said, for example, that a "neutral or astatic equilibrium characterizes a pure buckling and occurs at certain stages of an astatic action." The third term denotes what has been described as mixed buckling. Such action will be called heterostatic action. Loads producing orthostatic, astatic, and heterostatic actions, respectively, will be referred to as orthostatic, astatic, and heterostatic loads. A heterostatic load, therefore, can be resolved into two components: One an orthostatic load, the other an astatic load. From what has been stated previously, it follows that this investigation will deal mainly with general aspects of astatic action, and with heterostatic action. It will be shown that the cases of heterostatic action can be solved, by certain general formulas, in terms of corresponding cases of astatic and orthostatic actions.

2.—Remarks Concerning the Bibliography.—The classified Bibliography (Part VII) is not intended to include, in general, references dealing exclusively with straight columns which have constant cross-sections and which carry axial end loads only. This case is treated more or less extensively in almost any textbook on Mechanics of Materials. It is sufficient for the present purpose to point to Euler's analysis of columns, which dates back to 1757, and to the progress which has been marked by the introduction of such formulas as that of Rankine, the parabolic, and the straight-line formulas. A large amount of experimental work has been completed in this field. The deviations from Euler's formula are usually explained by the inevitable presence of small initial eccentricities. In fact, some of the textbooks* interpret the formulas mentioned as approximate representations of the secant formula, which applies to eccentric loading and which may be used when some definite "equivalent initial eccentricity" has been introduced.

The comparatively recent work by Kármán on straight columns is quoted in the Bibliography (Part VII, Section *B*). His experiments and analyses are noteworthy on account of the light they throw on the influence of stresses above the proportional limit. This matter is important in the analysis of the shorter columns. Especial attention is called to the treatment of S. Timoshenko of a great variety of cases of pure elastic buckling (Bibliography, Part VII, Section *A*), and also to the analysis of cylindrical shells by Goupil (Section *G*), in which a special case of the general formula for heterostatic action is derived.

II.—Formulas Applying to Columns Which Are Loaded by Axial and by Transverse Forces at the Same Time.

3.—Examples of Applications, Notation, and Current Approximate Formulas.—Straight columns loaded axially and at the same time bent by transverse

* See, for instance, J. E. Boyd, "Strength of Materials," 1917 Edition.

forces are found in many structures. Examples of such columns are as follows: Vertical columns which are loaded sidewise by wind pressure, the latter being transferred to the column either as concentrated forces or as a distributed load; horizontal or inclined compression members which are bent by their own weight; struts in aeroplanes which are bent by wind pressure; and the connecting rod in an engine which is bent by a dynamical action equivalent to a distributed load.

In this Article, and in the subsequent articles, the following notation will be used:

Let l = length of column;

EI = modulus of elasticity times moment of inertia of cross-section;

P = actual axial load;

Q = lowest critical axial load at which buckling would occur. When EI is constant throughout the length, Euler's formula gives: For columns with hinged ends, $Q = \pi^2 \frac{EI}{l^2}$; and for columns with fixed ends, $Q = 4\pi^2 \frac{EI}{l^2}$;

M' = resultant bending moment due to combined pure buckling action and bending, that is, to the combined axial and transverse loads. (M' is the moment in the "mixed" or "heterostatic" action);

M = bending moment when the transverse load acts alone (the moment in the case of orthostatic action).

Indices c and e are used to designate the moments at the center and at the ends, respectively. For instance, M_c = the moment at the center due to the transverse load alone. α, β, and μ are constants entering into the current approximate formulas. Further notation will be introduced in Article 7 (page 589).

Two current approximate formulas will be mentioned. In the first the ratio $\frac{\beta}{\alpha}$ enters. This ratio depends on the distribution of the load and on the end conditions. For example, columns with hinged ends and with a uniform transverse load have $\frac{\beta}{\alpha} = 9.6$; columns with hinged ends and with a concentrated load at the center have $\frac{\beta}{\alpha} = 12$; usually, $\frac{\beta}{\alpha}$ is not very far from 10. With the notation previously given, the formula may be written:*

$$M' = \frac{M}{1 - \frac{\alpha}{\beta}\frac{P}{E}\frac{l^2}{I}} \qquad (1)$$

In the other approximate formula, the constant, μ, occurs, which is stated as being not very far from unity. Sometimes, $\mu = 1$. The formula† is

$$M' = M\frac{\mu Q}{\mu Q - P} \qquad (2a)$$

* M. Merriman, "Mechanics of Materials," 11th Edition (1915), p. 256.

† A. Ostenfeld, "Teknisk Elasticitetslære", 3d Edition (1916), p. 456.

or, when $\mu = 1$,

$$M' = M \frac{Q}{Q - P} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2b)$$

When $\frac{\beta}{\alpha} = \mu \pi^2$, and when $Q = \frac{\pi^2 E I}{l^2}$, Formulas (1) and (2*a*) give identical results. These formulas furnish good approximations in many, but not all, cases.

4.—Proposed Design Formulas for Straight Columns with Uniform Cross-Section.—The theory leading to the formulas proposed here will be discussed later. The exact results are obtained as infinite series, which, however, converge so rapidly that for almost all practical purposes it is sufficient to include only the first two terms of each series. In formulas for design purposes, it is also permissible to simplify the constants entering into them by omitting the less significant decimals.

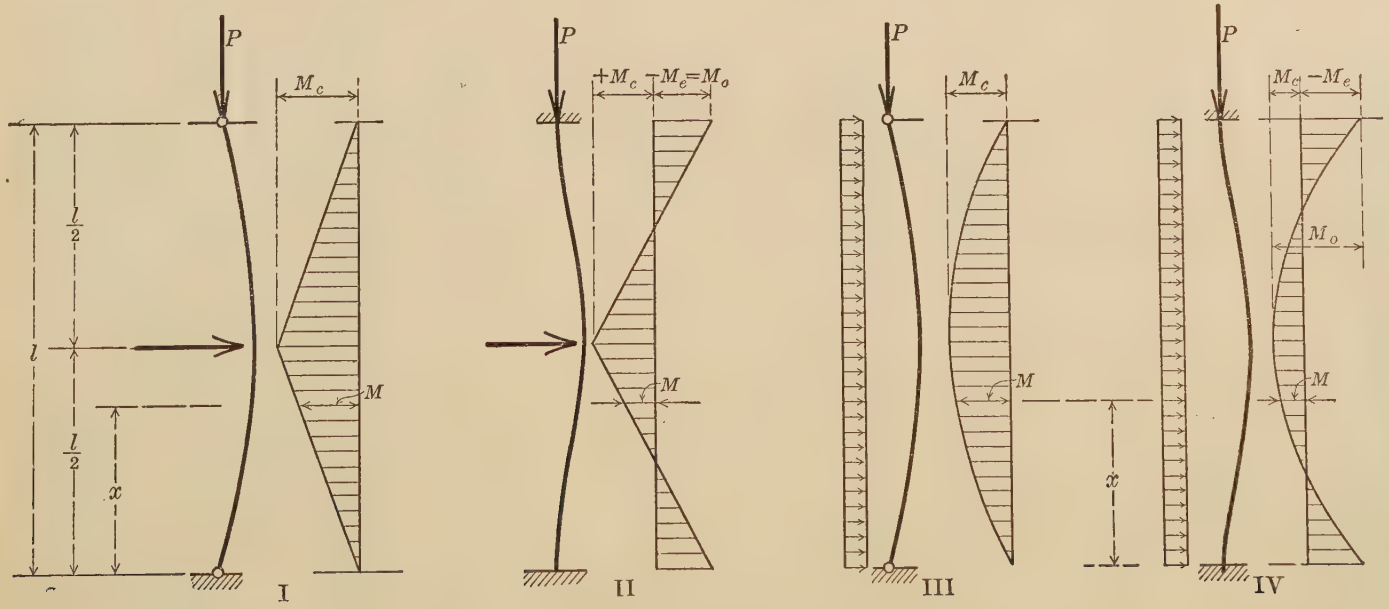
Four cases which have a particular interest, are shown in Fig. 2. The corresponding proposed design Formulas (3), (3*a*), (4), (4*a*), (5), and (6), are given in Table 1.

TABLE 1.—Proposed Design Formulas for Straight Columns.

EI is Assumed to be Constant Throughout the Length. The Limiting Errors Are for Positive *P* (Compression). Cases I, II, III, and IV Are Illustrated in Fig. 2.

		Concentrated load at center.	Uniformly distributed load.
Hinged-end columns: Moments at center, $Q = \frac{\pi^2 E I}{l^2}$.		Case I: $M_c' = M_c \left(1 + 0.8 \frac{P}{Q - P}\right)$ ..(3) (error less than 1.4%).	Case III: $M_c' = M_c \left(1 + 1.03 \frac{P}{Q - P}\right)$ ...(4) (error less than 0.2%) or, approximately, $M_c' = M_c \frac{Q}{Q - P}$ .....(4*a*) (error less than 3.2%).
Fixed-end columns; $Q = \frac{4\pi^2 E I}{l^2}$.	Moments at center.	Case II: $M_c' = M_c \left(1 + 0.8 \frac{P}{Q - P}\right)$ ..(3) (error less than 1.4%).	Case IV: $M_c' = M_c \left(1 + 1.2 \frac{P}{Q - P}\right)$ ....(5) (error less than 1.3%, and on the safe side; when $\frac{P}{Q} \leq 0.9$, the error does not exceed 1%).
	Moments at ends.	Case II: $M_e' = M_e \left(1 + 0.8 \frac{P}{Q - P}\right)$.(3*a*) (error less than 1.4%).	Case IV: $M'_e = M_e \left(1 + 0.65 \frac{P}{Q - P}\right)$ ...(6) (error less than 6.5% and on the safe side; for $\frac{P}{Q} \leq 0.5$, the error is less than 1%).

If a concentrated load and a distributed transverse load act at the same time, their combined effect can be computed by adding the values given by



COLUMNS WITH TRANSVERSE LOADS

FIG. 2.

the two separate formulas (in Table 1) which apply to these two separate loads; in other words, although the law of superposition does not apply to the total load, consisting of axial loads plus transverse loads, it does apply to the transverse loads alone as long as the axial load retains one constant value.

The formulas also apply, approximately, when P is negative, that is, when the axial load is a tension. In that case a catenary action takes place in which the axial load causes a reduction of the bending moments.

5.—Exact Formulas from Which the Preceding Design Formulas Have Been Derived.—Comparison with the Approximate Formulas.—Formulas (3) to (6), in Table 1, have been derived by simplification of exact expressions given as infinite series. These expressions apply both when P is positive, representing a compression, and when P is negative, representing a tension. P must not be equal to any of the critical loads.

In Cases I and II (Formulas (3) and (3*a*), for M'_c or M'_e and M_c or M_e), the column is loaded transversely at the center, and has fixed or hinged ends. The exact formula for the bending moments at the center in Cases I and II, and at the ends in Case II, is:

$$M'_c \text{ or } M'_e = M_c \text{ or } M_e\left[1 + \frac{8}{\pi^2}\frac{P}{Q-P} + \frac{8}{\pi^2}\frac{1}{3^2}\frac{P}{9\,Q-P} + \frac{8}{\pi^2}\frac{1}{5^2}\frac{P}{25\,Q-P} + \cdots\cdots + \frac{8}{\pi^2}\frac{1}{n^2}\frac{P}{n^2\,Q-P} + \cdots\cdots\right] = M_c \text{ or } M_e\left[1 + 0.811\,\frac{P}{Q-P} + 0.0901\,\frac{P}{9\,Q-P} + 0.0324\,\frac{P}{25\,Q-P} + \cdots\cdots\right] \quad (7)$$

in which $n = 1, 3, 5 \ldots\ldots$

In Case III (Formulas (4) and (4*a*), for the moment at the center, M'_c), the column has hinged ends and carries a uniform transverse load:

$$M'_c = M_c\left[1 + \frac{32}{\pi^3}\frac{P}{Q-P} - \frac{32}{\pi^3}\frac{1}{3^3}\frac{P}{9\,Q-P} + \cdots\cdots \pm \frac{32}{\pi^3}\frac{1}{n^3}\frac{P}{n^2\,Q-P} \mp \cdots\cdots\right] = M_c\left[1 + 1.032\,\frac{P}{Q-P} - 0.0382\,\frac{P}{9\,Q-P} + 0.0083\,\frac{P}{25\,Q-P} - \cdots\cdots\right] \quad (8)$$

in which $n = 1, 3, 5 \ldots\ldots$

Case IV (Formula (5)), gives the moment at the center, for a column with fixed end and uniform load:

$$M'_c = M_c\left[1 + \sum_{1,2,3,\ldots}^{n} \frac{12\,(-1)^{n-1}}{\pi^2\,n^2}\frac{P}{n^2\,Q-P}\right] = M_c\left[1 + 1.216\,\frac{P}{Q-P} - 0.304\,\frac{P}{4\,Q-P} + 0.1352\,\frac{P}{9\,Q-P} - 0.0760\,\frac{P}{16\,Q-P} + 0.0486\,\frac{P}{25\,Q-P} \cdots\right] \quad (9)$$

Case IV (Formula (6)), gives the moments at the ends for a fixed-end column, with uniform load:

$$M'_e = M_e \left[1 + \sum_{1,2,3\ldots}^{n} \sum_{1,2,3\ldots}^{m} \frac{6}{\pi^2 n^2} \frac{P}{n^2 Q - P} \right]$$

$$= M_e \left[1 + 0.608 \frac{P}{Q - P} + 0.1520 \frac{P}{4 Q - P} + 0.0676 \frac{P}{9 Q - P} + 0.0380 \frac{P}{16 Q - P} + 0.0243 \frac{P}{25 Q - P} \cdots \right] \quad \ldots\ldots\ldots (10)$$

The limiting errors indicated in Table 1 are found by comparison with the exact Formulas (7) to (10). Formula (6), derived from Formula (10), gives the greatest deviation. However, the error remains less than 1% when $\frac{P}{Q} \leq 0.5$. A quite satisfactory approximation is obtained by using the first three terms in Formula (10), which may then be written in the simplified form:

$$M'_e = M_e \left[1 + 0.6 \frac{P}{Q - P} + 0.2 \frac{P}{4 Q - P} \cdots \right] \ldots\ldots\ldots (10a)$$

Formulas (7) to (10) are rapidly converging series. For instance, $P = 0.5Q$, that is, the actual axial load equal to one-half of the critical axial load, gives the following values of $\frac{M'}{M}$, when two, three, or four, etc., terms of each series are included:

From Formula (7): 1.811, 1.816, 1.817, 1.817, ...
From Formula (8): 2.032, 2.030, 2.030, ...
From Formula (9): 2.216, 2.173, 2.181, 2,179, 2.180, ...
From Formula (10): 1.608, 1.630, 1.634, 1.635, 1.635, ...

The approximation obtained with the current approximate Formulas (1), (2*a*), and (2*b*), will now be examined in a typical case. Consider a hinged-end column loaded by a concentrated transverse force at the center. In that case, $\frac{\beta}{\alpha} = 12$. This value corresponds to $\mu = \frac{12}{\pi^2}$ in Formula (2*a*). The deviations from the exact values are then in percentage:

For $\frac{P}{Q}$ ..... =	0.2	0.5	0.9	1.0 (limiting case):
Deviations by using Formulas (1) and (2*a*) ..... =	— 1%	— 7%	— 54%	— 100%
Deviations by using Formula (2*b*) ... =	+ 4%	+ 10%	+ 20%	+ 23%

This comparison bears out the desirability of a revision of the current formulas. It must be stated, though, that, on account of the rapid variations

of M' when $\frac{P}{Q}$ approaches unity, the great deviations when $\frac{P}{Q} = 0.9$ or 1.0 are of comparatively less significance than the errors at smaller values of $\frac{P}{Q}$.

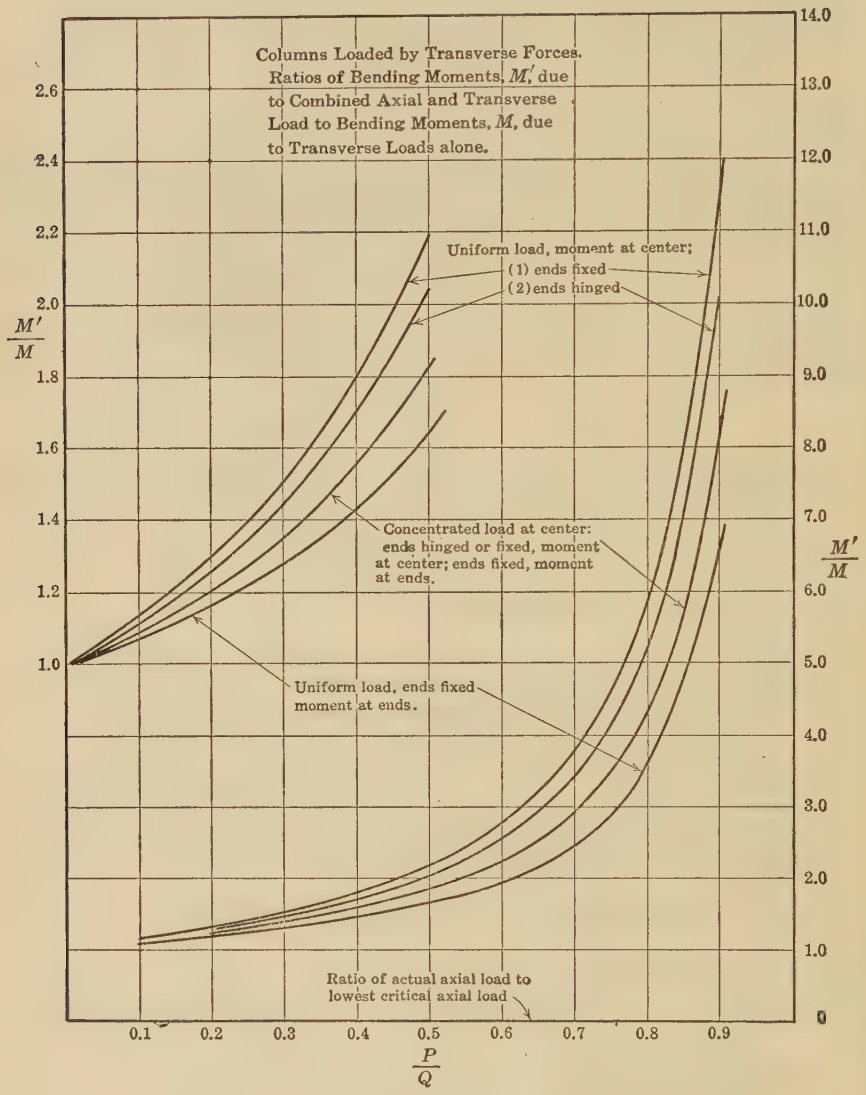


FIG. 3.

6.—Tabulated Numerical Results.—Graphs.—Table 2 is computed by Formulas (7) to (10). Fig. 3 represents the same data graphically.

7.—A More General Formula from Which the Preceding Formulas Have Been Derived.—The notation given in Article 3 is here supplemented, as follows:

Q_1, Q_2, $Q_3 \ldots . Q_n \ldots .$ = series of critical axial loads at which buckling (astatic elastic equilibrium) can take place. Q_1 is equal to Q in the preceding formulas. When both ends are fixed or both ends hinged, and when the cross-section is constant, then $Q_n = n^2 Q$. As mentioned in the Introduction, the neutral or astatic equilibria corresponding to the higher critical loads are unstable equilibria.

TABLE 2.—RATIOS, $\frac{M'}{M}$, OF TOTAL MOMENTS TO MOMENTS DUE TO TRANSVERSE LOADS ALONE FOR VARIOUS VALUES OF $\frac{P}{Q}$.

P = Axial Load ; for Hinged Ends, $Q = \frac{\pi^2 E I}{l^2}$; for Fixed Ends, $Q = \frac{4 \pi^2 E I}{l^2}$.

Load:	Concentrated at Center.		Distributed.		
Ends:	Hinged.	Fixed.	Hinged.	Fixed.	
Moment at:	Center.	Ends or center.	Center.	Center.	Ends.
$\frac{P}{Q} = 0$	1.000		1.000	1.000	1.000
0.1	1.091		1.114	1.129	1.073
0.2	1.205		1.257	1.290	1.163
0.3	1.351		1.442	1.502	1.277
0.4	1.546		1.686	1.782	1.428
0.5	1.817		2.030	2.18	1.636
0.6	2.223		2.545	2.78	1.95
0.7	2.901		3.405	3.78	2.46
0.8	4.25		5.13	5.79	3.48
0.9	8.31		10.29	11.87	6.52
1.0	∞		∞	∞	∞

M_1, M_2, $M_3 \ldots . M_n \ldots .$ = bending moments in the column which have the following properties: First, each of the moments, $M_1 \ldots \ldots . M_n \ldots \ldots$, is a function of the distance measured along the column, and may be represented by a diagram having the length of the column as a base; second, the diagram, M_n, is the moment diagram at a certain stage of the buckling or astatic action in which Q_n is the critical load; it is easily verified that if M_n is such a moment diagram, belonging to the buckling action the critical load of which is Q_n, then M_n multiplied by any constant factor will be another moment diagram belonging to the same buckling action, with the critical load Q_n; the greater this factor, the further progressed is the stage of this buckling; and, third, the moments, $M_1 \ldots . M_n \ldots .$, are so chosen by properly choosing the stages of buckling under each of the loads, $Q_1 \ldots . Q_n \ldots .$, respectively, that the total moment due to the transverse loads may be expressed by the following relation, which is to apply at all points of the column:

$$M = M_1 + M_2 + \ldots \ldots + M_n + \ldots \ldots \ldots \ldots \ldots (11)$$

It will be shown later that it is possible, in general, to do this. When it is possible to establish the relation, Formula (11), it may be done by mutiplying any set of moment diagrams which are proportional to M_1, M_2, $\ldots . M_n \ldots .$ and which are produced in astatic actions, by a proper set of factors.

The more general formula from which Formulas (7) to (10) have been derived, will now be indicated. The derivation of the formula will follow later (Part IV). The formula has a wide field of application, but will appear in Part IV as a special case of a still more general formula which applies to any heterostatic action. The formula applies primarily to those cases of axially and transversally loaded straight columns in which the supports do not absorb any elastic energy, that is, it applies, for example, when the ends are fixed or simply supported (hinged), or when one end is free and the other end fixed, and also when there are inelastic supports at intermediate points. The formula also applies, in general, even when the supports are elastic, but in that case some qualifications must be introduced as to the make-up of the functions, $M_1 \ldots\ldots M_n \ldots\ldots$ This result will follow from the general theory in Part IV. The formula applies whether the cross-section is constant or varies, or whether the axial load is applied at the ends or at other points. The formula is:

$$M' = M + M_1 \frac{P}{Q_1 - P} + M_2 \frac{P}{Q_2 - P} + \ldots\ldots + M_n \frac{P}{Q_n - P} + \ldots\ldots (12)$$

The essential difficulty in the use of Formula (12) lies in the determination of the functions, $M_1 \ldots\ldots M_n \ldots\ldots$ The writer will now show how these difficulties are overcome in various specific cases.

8.—Deriving the Specific Formulas from the General Formula.—The formulas applying to the four cases in Fig. 2 will now be derived from the general Formula (12). The cross-section is again assumed to be constant, and the same typical method will be seen to apply in all four cases.

Case I.—Hinged-end column, carrying a concentrated transverse load at the center (Fig. 2 (I)): The formula which is to be derived is Formula (7). The derivation of the approximate Formula (3) from Formula (7) was indicated in Article 5.

The first step is to find the deflections and the bending moments in the buckling actions in which $Q_1, Q_2 \ldots\ldots Q_n \ldots\ldots$ are the critical loads. To the critical load, Q_n, corresponds a buckling curve or curve of deflections consisting of n half waves. From the theory of columns, as given in texts on Mechanics of Materials, this curve is known to be a sine wave. The bending moments are found as the second derivatives of the deflections multiplied by the constant factor, $\pm EI$. It follows also that the moment diagram, M_n, will be shaped as a sine curve with n half waves. Let x be the distance measured along the column from the lower end; then the equation for M_n may be written:

$$M_n = C_n \sin \left(\frac{n \pi x}{l}\right) \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

where C_n is a constant. The bending moment, M, due to the transverse load alone, should now be expressed in the form of the series, $M = \Sigma M_n$ (Formula (11)). M is given by the triangular moment diagram in Fig. 2 (I). The series, ΣM_n, is a trigonometric series. From the theory of Fourier series* it is known that any diagram can be represented by an infinite trigonometric series (Fourier series). It is possible, therefore, to find a set of constants, C_n,

* For instance, Byerly, "Harmonic Functions", 4th ed. (1906).

which, introduced into Formula (13), make $M = \Sigma M_n$. From the theory of Fourier series, we have:

$$C_n = \frac{2}{l} \int_0^l M \sin\left(\frac{n \pi x}{l}\right) d x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Substituting the values for M given in Fig. 2 (I), integrating, and forming the expressions for M_n, we find:

$$\text{for } n \text{ even}, \quad M_n = 0$$

$$\text{for } n \text{ uneven}, \quad M_n = (-1)^{\frac{n-1}{2}} \frac{8 M_c}{\pi^2 n^2} \sin \frac{n \pi x}{l}.$$

Hence, the series for M may be written:

$$M = \Sigma M_n = M_c \left[\frac{8}{\pi^2} \sin \frac{\pi x}{l} - \frac{8}{\pi^2} \frac{1}{3^2} \sin \frac{3 \pi x}{l} + \frac{8}{\pi^2} \frac{1}{5^2} \sin \frac{5 \pi x}{l} - \ldots\ldots\ldots \right]$$

The expressions for M_n appear in this series with the constant factor, M_c. By substituting these expressions for M_n in Formula (12) and by putting $Q_n = n^2 Q$, the following formula is found for the moment, M', produced at any point, under the combined influence of the transverse and the axial load:

$$M' = M + M_c \left[\frac{8}{\pi^2} \frac{P}{Q - P} \sin \frac{\pi x}{l} - \frac{8}{\pi^2} \frac{1}{3^2} \frac{P}{9 Q - P} \sin \frac{3 \pi x}{l} \right.$$

$$\left. + \frac{8}{\pi^2} \frac{1}{5^2} \frac{P}{25 Q - P} \sin \frac{5 \pi x}{l} - \ldots\ldots\ldots\ldots \right]$$

We are particularly interested in the maximum moment, which occurs at the center. At that point, n uneven gives $\sin \frac{n \pi x}{l} = \pm 1$. The substitution of these values, ± 1, in the foregoing equation for M' leads directly to Formula (7) which was to be derived.

The other cases, II, III, and IV, in Fig. 2, for which special formulas have been given, can be treated in a similar manner. In all cases, the moment diagrams, M_n, in pure buckling action (astatic action) are trigonometric diagrams, and the series, ΣM_n, are Fourier series which represent the triangular and parabolic moment diagrams given in Fig. 2. Each of these cases will now be discussed.

Case II.—Column with fixed ends, and with a transverse load at the center: In the deflected curve, the quarter-points will be points of inflection. The column will be in equilibrium when the middle half is bent according to the laws applying to the whole length in Case I, and the outer quarters of the length will be bent into the same shape as the middle quarters. It follows that Formula (7) applies also to this case and expresses both the moments at the ends and at the center.

Case III.—Hinged ends, uniform transverse load: Formulas (13) and (14) apply also to this case, only, the values of M are to be taken from the parabolic moment diagram. M_n is found to be:

$$\text{for } n \text{ even}, \quad M_n = 0$$

$$\text{for } n \text{ uneven}, \quad M_n = \frac{32 M_c}{\pi^3 n^3} \sin \frac{n \pi x}{l}$$

The substitution of these values, with $x = \frac{l}{2}$, in the general Formula (12), leads to Formula (8).

Case IV.—Fixed ends, uniform transverse load: When the distance, x, is measured from the center, we find:

$$M = M_0 \left(\frac{1}{3} - \frac{4\,x^2}{l^2}\right)$$

where,

$$M_0 = 3\,M_c = -1.5\,M_e$$

M_n takes the form:

$$M_n = C_n \cos n \left(\frac{2\,\pi\,x}{l}\right)$$

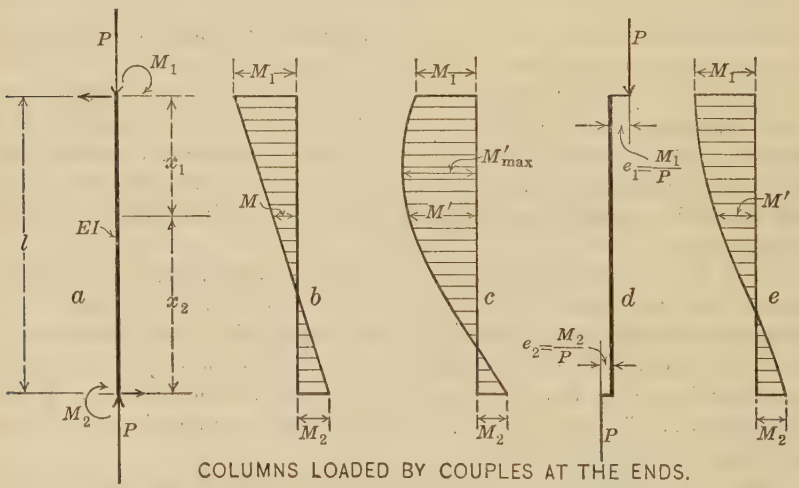
where, by the theory of Fourier series:

$$C_n = \frac{2}{l} \int_{-\frac{l}{2}}^{+\frac{l}{2}} M \cos n \left(\frac{2\,\pi\,x}{l}\right) d\,x$$

After integration, we find in this manner:

$$M_n = (-1)^{n-1} \frac{4\,M_0}{\pi^2\,n^2} \cos \frac{n\,2\,\pi\,x}{l}$$

Substitution of these values in the general Formula (12), with $x = 0$ and $x = \frac{l}{2}$, leads to Formulas (9) and (10).



COLUMNS LOADED BY COUPLES AT THE ENDS.

Fig. 4.

9.—*Columns Loaded by Couples at the Ends.*—The column in Fig. 4 (*a*) carries a central axial load, P, and at the same time is loaded by the couples, M_1 and M_2, which act at the ends, and are considered as positive in clockwise direction. The cross-section is assumed to be constant. This problem may be

treated by means of the general Formula (12) in the same manner as the preceding cases, but the particular conditions of the present problem allow a rather simple direct solution, which will now be indicated.* Let the notation for distances be as shown in Fig. 4 (*a*). The moment diagram produced by the couples alone is shown in Fig. 4 (*b*). In the combined bending and buckling action the resultant moment diagram may shape itself as shown in Fig. 4 (*c*). Evidently, the same action might be produced by the forces, P, alone, namely, when they are applied with the eccentricities indicated in Fig. 4 (*d*).

The particular object is to find the maximum values of the moments, M'. This may be done as follows: M' is expressed in terms of M_1, M_2, x_1, and P, and the deflections, y. The expression formed in this manner is substituted in the flexure equation, $E\,I\,\frac{d^2y}{d\,x_1^2} = -\,M'$, which is then integrated, whereupon M' is found by a double differentiation. The quantities are introduced:

$$Q = \frac{\pi^2\,EI}{l^2} = \text{lowest critical axial load when ends are hinged;}$$

and,

$$k = \frac{P}{Q}$$

The formula found for M' at any point is then:

$$M' = \frac{M_1 \sin \pi \left(\frac{x_2}{l}\right)\sqrt{k} - M_2 \sin \pi \left(\frac{x_1}{l}\right)\sqrt{k}}{\sin \pi \sqrt{k}} \quad \ldots\ldots\ldots (15)$$

M' becomes maximum or minimum when $\frac{d\,M'}{d\,x_1} = 0$. Since, $dx_1 = -\,dx_2$, the derivative becomes zero when,

$$\cos \pi \left(\frac{x_2}{l}\right)\sqrt{k} + \left(\frac{M_2}{M_1}\right) \cos \pi \left(\frac{x_1}{l}\right)\sqrt{k} = 0 \quad \ldots\ldots\ldots (16)$$

Formula (16) is adaptable to graphical solution.

It may happen that no solution of Formula (16) occurs within the length of the column. In that case, if M_2 is numerically smaller than M_1, the moment, M', will decrease on the whole length of the column from the top to the bottom. The case is represented in Fig. 4 (*e*), where the value, M_1, of the end moment is the greatest value of M'. The case occurs when the axial load, P, is comparatively small, that is, when k is comparatively small. The limiting case is found by inserting $x_1 = 0$, $x_2 = l$ in Formula (16), whereby we find,

$$\cos \pi \sqrt{k} + \frac{M_2}{M_1} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

For each value of $\frac{M_2}{M_1}$ between $+\,1$ and $-\,1$, a corresponding limiting value of $k = \frac{P}{Q}$ may be found from Formula (17). At and below this limiting

* For the derivation of the deflected curve see A. Ostenfeld, "Teknisk Elasticitetslære", 3d ed. (1916), p. 452. The case is important with regard to the question of shearing stresses in columns (*ibid*, p. 453).

value, we have max. $M' = M_1$. Above the limiting value of k, the values of x_1 and x_2 found from Formula (16), are to be substituted in Formula (15), thereby giving a value of max. M'.

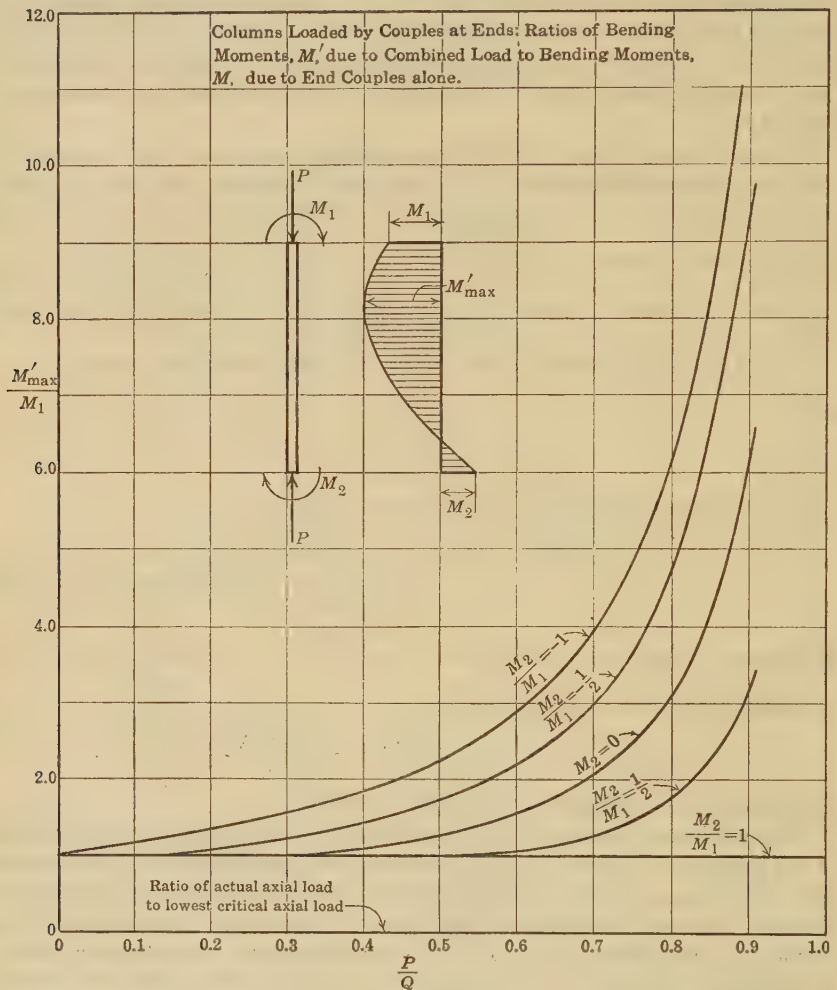


FIG. 5.

Three special cases will be examined, namely, $M_2 = -M_1$, $M_2 = 0$, and $M_2 = M_1$, respectively. First, assume $M_2 = -M_1$: In this case the moment due to the end couples alone is constant throughout the length. The case may be produced by eccentric loading with the same eccentricity, to the same side, at the top, and at the bottom. Formula (16) gives, in this case, $x_1 = x_2 = \frac{l}{2}$,

that is, as would be expected, the greatest moment occurs at the center. Formula (15) gives then:

$$\text{max. } M'_{M_2 = -M_1} = \frac{2 M_1 \sin \frac{\pi}{2}\sqrt{k}}{\sin \pi \sqrt{k}} = M_1 \sec \frac{\pi}{2}\sqrt{k} \quad \ldots\ldots (18)$$

This equation is a form of the secant formula which applies to eccentric loading. The second special case, $M_2 = 0$, occurs when the column is hinged at the lower end. Formulas (15) and (16) give:

$$\text{max. } M'_{M_2 = 0} = \frac{M_1}{\sin \pi \sqrt{k}} \text{ at point } x_2 = \frac{l}{2\sqrt{k}} \quad \ldots\ldots (19)$$

In the third special case, $M_1 = M_2$, Formula (17) gives $k = 1$ as the limiting value. It follows that for all values of P less than the critical load, Q, the greatest moment occurs at the ends, that is,

$$\text{max. } M'_{M_2 = M_1} = M_1 \quad \ldots\ldots (20)$$

Table 3 gives solutions of the preceding formulas for various values of $\frac{M_2}{M_1}$ and for values of $\frac{P}{Q}$ between 0 and 1. The same data are presented graphically in Fig. 5.

TABLE 3.—Columns Bent by Couples at the Ends.

$\frac{x_1}{l}$ = Ratio of Distance from Upper End to Point of Maximum Moment;

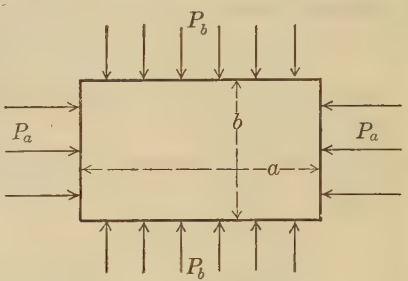
$\frac{M'}{M_1}$ = Ratio of Maximum Moment to Moment at Top;

$Q = \frac{\pi^2 EI}{l^2}$; otherwise, the notation is as given in Fig. 5.

$\frac{P}{Q}$	$M_2 = -M_1$		$M_2 = -\frac{1}{2}M_1$		$M_2 = 0$		$M_2 = \frac{1}{2}M_1$		$M_2 = M_1$	
	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$
0.0	.....	1.00	0	1	0	1	0	1	0	1
0.1	0.5	1.14	0	1	0	1	0	1	0	1
0.111	.....	.....	0	1.00	.....	.....	.....	.....	.....	.....
0.2	0.5	1.31	0.23	1.06	0	1	0	1	0	1
0.25	.....	.....	.....	.....	0	1.00	.....	.....	.....	.....
0.3	0.5	1.54	0.34	1.20	0.09	1.01	0	1	0	1
0.4	0.5	1.83	0.39	1.41	0.21	1.09	0	1	0	1
0.444	.....	.....	.....	.....	.....	.....	0	1.00	.....	.....
0.5	0.5	2.25	0.43	1.72	0.29	1.26	0.06	1.01	0	1
0.6	0.5	2.89	0.45	2.18	0.35	1.54	0.16	1.08	0	1
0.7	0.5	3.94	0.47	2.96	0.40	2.04	0.25	1.25	0	1
0.8	0.5	6.06	0.48	4.55	0.44	3.07	0.33	1.69	0	1
0.9	0.5	12.5	0.49	9.35	0.47	6.25	0.42	3.20	0	1
1.0	0.5	∞	0.50	∞	0.50	∞	0.50	∞	.....	∞

III.—Slabs and Similar Structures.

10.—Formulas Applying to Rectangular Homogeneous Slabs.—A rectangular slab may buckle under the influence of loads at the edges acting in the original plane of the slab. Two examples of this kind were mentioned in the Introduction (Fig. 1 (*K*) and Fig. 1 (*L*)). In the one case, the load was perpendicular to the edge; in the other, it was parallel to the edge. Until the deflections have become quite large, the action is a typical astatic elastic action (pure buckling). Loads from the side, perpendicular to the slab, acting alone, produce orthostatic action, that is, ordinary flexure with the deformations proportional to the loads. A combination of the end loads in the plane of the slab and the side loads produces mixed or heterostatic action. When the frames of a ship are rather widely spaced, such actions may become important in the shell (deck, sides, and bottom). Cases of buckling of rectangular slabs have been analyzed by Messrs. G. H. Bryan, S. Timoshenko, and others, and reference is made to the treatments by these authors (Bibliography, Part VII, Sections *A* and *J*). The material presented by the writer will include some cases of buckling or astatic action as well as some cases of mixed or heterostatic action.



SLAB WITH BUCKLING LOADS.
Fig. 6.

Let (Fig. 6), a = longer span;
b = shorter span;
$\alpha = \frac{b}{a}$

EI = modulus of elasticity times moment of inertia of cross-section per unit width;

K = Poisson's ratio of lateral contraction to longitudinal elongation when a longitudinal load acts alone;

w = uniformly distributed load per unit area, perpendicular to the surface of the slab;

P_a = uniformly distributed compressive end load per unit width, acting at the short edges in the direction of the long span. A negative P_a represents tension;

P_b = uniformly distributed compressive end load per unit width, acting at the long edges in direction of the short span;

Q_a and Q_b = critical values of the end loads found by increasing the original values of P_a and P_b by the same ratio until buckling can take place without the aid of the surface load, w;

M = moment in slab per unit width in the orthostatic action, in which the surface load, w, is the only external load;

M' = moment in the mixed or heterostatic action in which end loads, such as P_a and P_b, act at the same time as the surface load, w;

$M'' = M' - M$ = increase of moment produced by application of the end load when the surface load is already acting;

M_b, M'_b, M''_b = values of M, M', and M'' at the center of the slab, for bending in the short span, b (that is, in an element of section perpendicular to the short span, b);

m, n = positive integers.

The writer will confine himself to cases in which the only end loads in the plane of the slab are those perpendicular to the edges, such as, P_a and P_b in Fig. 6. It is assumed that each rectangular panel can be treated separately, and, accordingly, only one single independent panel will be considered. The slab is assumed to be homogeneous, isotropic, and sufficiently thin so that it may begin to buckle without exceeding the elastic limit. It is also assumed that the deflections have not progressed so far as to affect the uniformity of the distribution of the pressures in the plane of the slab. The slab under consideration has simple supports along the four edges, and these supports hold the edges in the original plane of the slab.

The writer will again follow the plan of presenting results first. The derivations follow later, in Parts IV and V.

A.—Astatic Action, Buckling Due to End Loads Alone, When the Slab is Simply Supported.

An astatic (neutral) equilibrium may occur when the end loads, Q_a and Q_b, satisfy the condition:

$$\frac{m^2 Q_a}{a^2} + \frac{n^2 Q_b}{b^2} = \frac{\pi^2 E I}{1 - K^2}\left[\frac{m^2}{a^2} + \frac{n^2}{b^2}\right]^2 \quad \ldots\ldots\ldots\ldots (21)$$

where $m, n = 1, 2, 3 \ldots\ldots$*. The lowest critical values of Q_a and Q_b have the greatest interest, as they are the values at which the actual buckling would take place if the load was increased gradually from zero. In buckling caused by the lowest critical loads, one of the integers, m and n, is always equal to 1. The values of m and n, and the corresponding solutions of Formula (21), will now be indicated in some definite cases.

Case 1.—Square slab, $a = b$:

a.—Q_b positive, $Q_a = 0$ (compression in direction of one span only):

$$m = n = 1$$

Formula (21) gives,

$$Q_b = \frac{4 \pi^2 E I}{(1 - K^2) b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

b.—Q_a and Q_b both positive (both compressive loads):

$$m = n = 1$$

In this case, Formula (21) becomes,

$$Q_a + Q_b = \frac{4 \pi^2 E I}{(1 - K^2) b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

* Formula (21) was given by G. H. Bryan, *Proceedings*, London Math. Soc., v. 22 (1890), p. 57; see, also, A. E. H. Love, "Mathematical Theory of Elasticity" (1906), p. 529.

c.—Q_b positive, $Q_a = -Q_b$ (that is, Q_a is a tension equal in magnitude to the compression, Q_b). This case is of some importance because it represents the condition of pure shear in the directions of the diagonals:

$$m = 1, n = 2$$

Substitution of these values in Formula (21) gives:

$$Q_b = \frac{25\,\pi^2\,E\,I}{3\,(1 - K^2)\,b^2} \qquad (24)$$

d.—Q_b positive, $Q_a = -2Q_a$:

$$m = 1, n = 2;\ Q_b = \frac{25\,\pi^2\,E\,I}{2\,(1 - K^2)\,b^2}$$

e.—General case, Q_b positive, ratio $\frac{Q_a}{Q_b}$ any value:

$$m = 1;\ Q_b = \frac{\pi^2\,E\,I}{(1 - K^2)\,b^2}\,\frac{(n^2 + 1)^2}{n^2 - \left(-\frac{Q_a}{Q_b}\right)} \qquad (25)$$

$$n = 1 \text{ when } -\frac{Q_a}{Q_b} < 0.43$$

$$n = 2 \text{ when } 0.43 < -\frac{Q_a}{Q_b} < 2.33$$

$$n = 3 \text{ when } 2.33 < -\frac{Q_a}{Q_b} < 5.30$$

Case 2.—Rectangular slab, $a > b$. $Q_a = 0$, that is, the load is in the direction of the short span only:

$$m = n = 1;\ Q_b = \frac{\pi^2\,E\,I\,(1 + \alpha^2)^2}{(1 - K^2)\,b^2} \qquad (26)$$

It is noted that the limiting case, $\alpha = 1$, leads to Formula (22). $\alpha = 0$, that is, $a = \infty$, and Poisson's ratio, $K = 0$, gives the usual Euler formula for columns, $Q = \frac{\pi^2\,E\,I}{b^2}$.

Case 3.—Rectangular slab, $a > b$. $Q_b = 0$, that is, the load is in the direction of the long span only:

$$n = 1;\ Q_a = \frac{\pi^2\,E\,I}{(1 - K^2)\,b^2}\,\frac{(m^2\,\alpha^2 + 1)^2}{m^2\,\alpha^2} \qquad (27)$$

When $\frac{1}{\alpha}$ is an integer, then $m = \frac{1}{\alpha}$. Otherwise, m = either the nearest integer above, or the nearest integer below, $\frac{1}{\alpha}$. This result may be proved by considering m as varying continuously; then Q_a in Formula (27) is found to have a minimum when $m = \frac{1}{\alpha}$. This result means physically that in the buckling of the center line of the long span, the half-wave length (distance between points of inflection) has a tendency to become equal to the shorter

span of the slab. Under any circumstance, $m\ \alpha = 1$ substituted in Formula (27) defines a lower limit of the critical load, namely,

$$Q_a = \frac{4\ \pi^2\ E\ I}{(1 - K^2)\ b^2} \qquad (27a)$$

In most cases Formula (27*a*) would replace Formula (27) with good approximation.

Case 4.—General case: Rectangular slab, $b < a$, $\frac{Q_a}{Q_b}$ = any value. The general Formula (21) is to be used. It was stated that in order to obtain the lowest critical loads one of the integers, m and n, must always be equal to 1. The following rule applies:

$$m = 1 \text{ when } \frac{Q_a}{Q_b} < 0.5,\ Q_b > 0;$$
$$n = 1 \text{ when } Q_a \geqq 0.$$

The general Formula (21) is the equation of a straight line in the coordinate system, Q_a, Q_b (Fig. 7). By keeping m constant, but varying n, a set of straight lines is obtained all of which are tangents to the same parabola with the axis parallel to the Q_a-axis. In the same way, variation of m while n is kept constant, gives a set of tangents to a parabola with the axis parallel to the Q_b-axis. Two such sets of tangents, one for $m = 1$ and one for $n = 1$, form a polygon which defines the lowest critical values, Q_a and Q_b, for all ratios, $\frac{Q_a}{Q_b}$. The two parabolas pass through the origin with their tangents making angles of 45° with the axes. The parabolas are open backward and downward. Fig. 7 shows the parabolas for the cases of the square slab, $a = b$, and the rectangular slab, $a = 2\ b$. The vertices of the polygons represent conditions under which two different buckling waves may be superimposed, one on the other, so that a double, neutral equilibrium occurs at those particular loads.

B.—Orthostatic Flexure, Bending Without End Loads in the Plane of the Slab, the Slab Being Simply Supported Along the Four Edges.

The surface load, w, is acting, but there are no pressures from the ends. It will be assumed temporarily that Poisson's ratio, K, of lateral contraction to longitudinal stretch is zero. The moment at the center in Span b is then found to be given approximately by the following formula:

$$M_b = \frac{\frac{1}{8}\ w\ b^2}{1 + \alpha^2 + 1.4\ \alpha^4} \qquad (28)$$

In a slab which is square or nearly square greater moments occur along and across the diagonals at the corners than at the center. Formula (29) may be used with advantage in many cases. It gives a reasonable approximation to the maximum moment in the slab. In slabs which are nearly square, it gives somewhat too low stresses, but a slight local yielding at the corners

will re-distribute the stresses until values almost similar to those given by the formula are obtained:*

$$M = \frac{\frac{1}{8} w b^2}{1 + 2\alpha^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

It is noted that $\frac{b}{a} = \alpha = 1$ would give $M = \frac{1}{24} w b^2$, which is a value often used for square slabs. When Poisson's ratio, K, is different from zero, Formulas (28) and (29) indicate the product of the curvature (second derivative of the deflections) times the stiffness ratio, $\frac{E I}{(1 - K^2)}$. According to the "strain theory" of rupture—St. Venant's theory—$E I$ times the curvature will be the moment quantity indicating the tendency to rupture. Accordingly, if this theory could be assumed to hold, Formulas (28) and (29) multiplied by $(1 - K^2)$ could be used as "theoretical moments" measuring the nearness of rupture.

C.—Heterostatic (Mixed) Action.

Poisson's ratio, K, is again assumed to be equal to zero. The difference brought about by Poisson's ratio not being zero is of the same nature as that described for the case of orthostatic flexure. An approximate formula will be indicated for the additional bending moment, M''_b, in Span b at the center. The approximation is close only when the combination $m = n = 1$ in Formula (21) leads to the lowest critical end loads, Q_a and Q_b. Formula (30) is similar to the approximate formulas mentioned previously for columns. It is derived from the general theory which follows later (Part IV), with use made of a solution of a differential equation of flexure given in Part V (Formula (150)). The formula is:

$$M''_b = \frac{16}{\pi^4} \frac{w b^2}{(1 + \alpha^2)^2} \frac{P_a}{Q_a - P_a} = \frac{16}{\pi^4} \frac{w b^2}{(1 + \alpha^2)^2} \frac{P_b}{Q_b - P_b} \quad \ldots\ldots (30)$$

This value is to be added to M_b in Formula (28), and gives thereby the combined moment, M'_b. It will be on the safe side to compute a value of the maximum moment, M', at any point by adding M''_b to the maximum orthostatic moment, M. Taking M from Formula (29) and substituting $\frac{16}{\pi^4} = 0.164$, the combined maximum moment, one finds:

$$M'_{\max.} = M + M'' = \frac{\frac{1}{8} w b^2}{1 + 2\alpha^3} + \frac{0.164\, w b^2}{(1 + \alpha^2)^2} \frac{P_a}{Q_a - P_a} \quad \ldots\ldots (31)$$

* Formula (28) follows rather closely values given by A. Nádai, in "Die Formänderungen und Spannungen von rechteckigen elastischen Platten", *Mitteilungen* über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Vol. 170–171, table, p. 34, provided correction is made for the effect of Poisson's ratio. A formula similar to Formula (28) is indicated in a note on the design of reinforced concrete, issued by the French Council on Bridges and Roads, entitled, "Calcul des Hourdis en Béton Armé", *Annales des Ponts et Chaussées*, 1912, IV, pp. 469–529 (see in particular, p. 474). Mesnager indicates a formula which is similar to Formula (29), see his memoir, "Moments et Flèches des Plaques Rectangulaires Minces Portant une Charge Uniformément Répartie", *Annales des Ponts et Chaussées*, 1916, III, pp. 313–438 (see in particular p. 436 and Plate VII). The derivation of the differential equation of flexure, on the solutions of which these analyses depend, is indicated in Part V, Article 31, of this investigation.

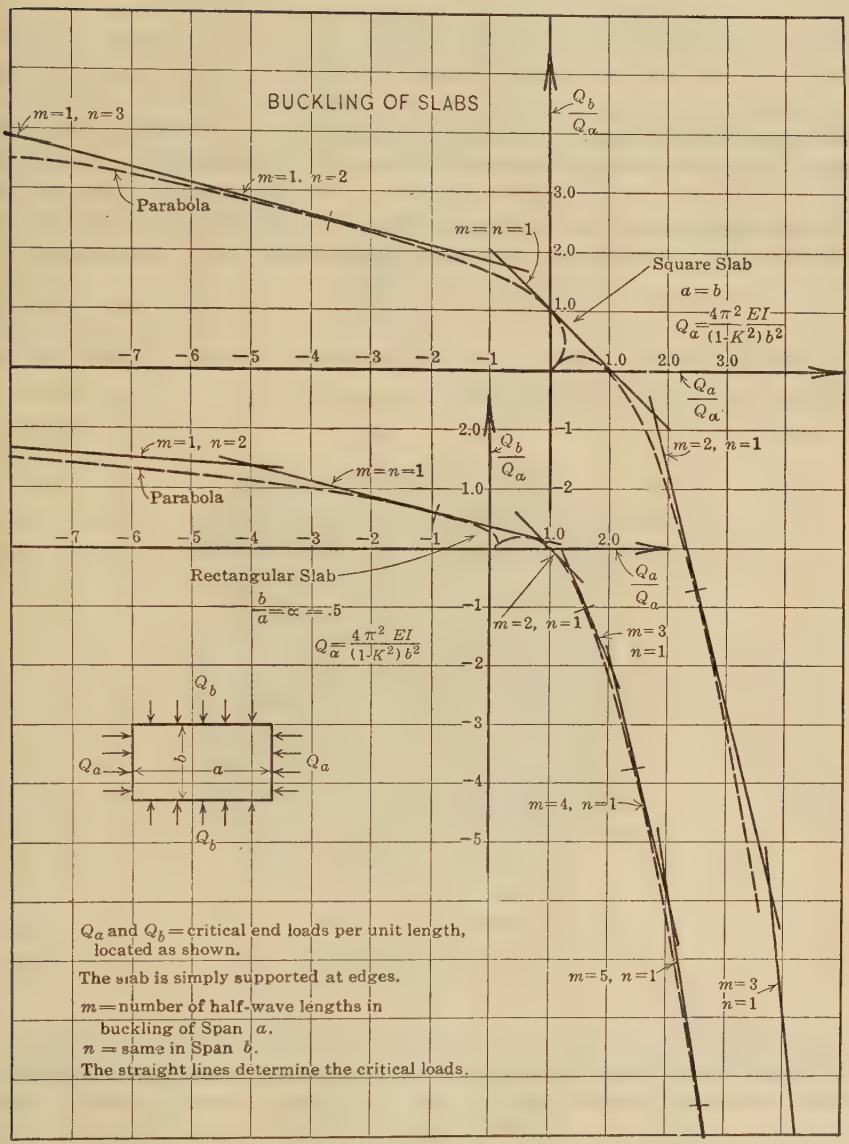

BUCKLING OF SLABS
$\frac{Q_b}{Q_a}$
m=1, n=3
m=1, n=2
Parabola
m=n=1
3.0
2.0
1.0
Square Slab
a=b
$Q_\alpha=\frac{4\pi^2 EI}{(1+K^2)b^2}$
−7 −6 −5 −4 −3 −2 −1
1.0 2.0 3.0
$\frac{Q_a}{Q_a}$
m=1, n=2
Parabola
m=n=1
2.0
1.0
−1
−2
m=2, n=1
−7 −6 −5 −4 −3 −2 −1
1.0 2.0
Rectangular Slab
$\frac{b}{a}=\alpha=.5$
$Q_\alpha=\frac{4\pi^2 EI}{(1+K^2)b^2}$
m=2, n=1
m=3
n=1
−1
−2
−3
−4
−5
Q_b
Q_a
b
a
Q_a
Q_b
m=4, n=1
m=5, n=1
m=3
n=1
Q_a and Q_b = critical end loads per unit length, located as shown.
The slab is simply supported at edges.
m = number of half-wave lengths in buckling of Span a.
n = same in Span b.
The straight lines determine the critical loads.

FIG. 7.

Formula (31) applies with the same limitations as Formula (30). In the case of a square slab, we have $\alpha = 1$. In that case, Formula (31) gives:

$$M'_{\max.} = \frac{1}{24} w b^2 + 0.041 \; w b^2 \frac{P_a}{Q_a - P_a}$$

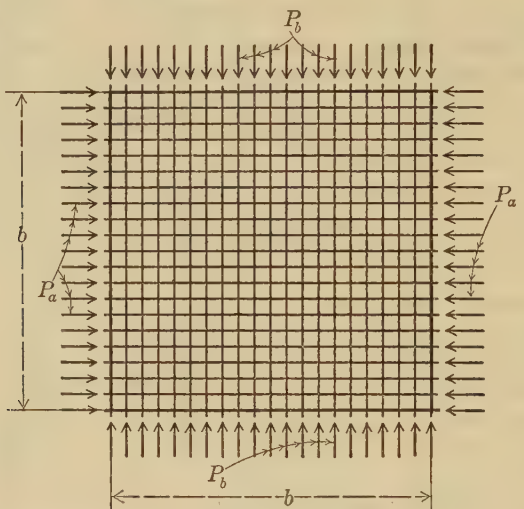
that is, with sufficiently close approximation,

$$M'_{\max.} = \frac{w b^2}{24} \frac{Q_a}{Q_a - P_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

When one of the integers m and n is different from 1, Formulas (30), (31), and (32), with Q_a and Q_b taken as the lowest critical loads, will lead to values which are in most cases decidedly higher than the actual values. However, in all such cases, as well as in those in which Formulas (30) to (32) apply, Formula (33) will be on the safe side, or only slightly on the unsafe side:

$$M' = M \frac{Q_a}{Q_a - P_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Formula (33) has the same form as the current approximate Formula (2*b*) in Article 3 and Formula (4*a*) in Article 4.



DOUBLE SYSTEM OF CROSSING BEAMS WITH BUCKLING LOADS.

FIG. 8.

11.—Formulas Applying to a Double System of Crossing-Beams Connected at Cross-Points.—Fig. 8 shows two systems of beams crossing one another at right angles and arranged in a square. The beams are simply supported at the edges of the square so that these edges must remain in their original plane. The beams are connected at the cross-points so as to make the deflections of the two beam systems the same without causing any torsional moments in the beams. A great number of beams are assumed, uniformly spaced and with uniform stiffness. The end loads on each beam system are evenly distributed, and are transmitted directly to each beam as axial loads, so as to give a

constant total axial pressure in any section of any beam. The structure resembles a slab, but the difference lies in the lack of torsional resistance in the double-beam system. This quality is demonstrated by the fact that one could lift one of the corners out of the original plane, and thereby produce a double curved surface, without putting stresses in any of the beams. It may be interesting to investigate this case for the purpose of comparison with the homogeneous slab. A square concrete slab with reinforcing bars parallel to the edges has properties intermediate between those of the homogeneous slab and those of the double-beam system, and the resistance of two-way reinforced concrete against buckling might be estimated by comparison with the two extreme cases.

The same notation will be used as in Article 10. The loads, moments, and the stiffness factor, EI, are indicated per unit width of section. It will be assumed that the spacing of the beams is so close that the area elements of the structure may be considered as continuous.

A.—Astatic Action.

The general formula for buckling, to be compared with Formula (21) in Article 10, is

$$m^2 Q_a + n^2 Q_b = \frac{\pi^2 E I (m^4 + n^4)}{b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

When both Q_a and Q_b are positive, the buckling at the lowest load is represented by $m = n = 1$. In this case, Formula (34) becomes:

$$Q_a + Q_b = \frac{2 \pi^2 E I}{b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Formula (35) should be compared with Formula (23) which applies to the homogeneous slab. When EI is the same in the two formulas, and K in Formula (23) is zero, Formula (35) is seen to give only one-half the value of the critical loads found by Formula (23), that is, under those circumstances, elimination of the torsional resistance in a square slab cuts the buckling resistance in two. The resistance to a load in one direction only is still twice the critical load which would be found if the supporting edges parallel to the load were removed.

B.—Orthostatic Flexure.

The surface load, w, acting alone, gives a maximum bending moment, occurring at the center, equal to

$$M_{\max.} = 0.077\ w\ b^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

(to be compared with $0.037\ wb^2$ in the square homogeneous slab, as given by Formula (28) with $\alpha = .1$). The method of deriving Formula (36) will be indicated in Part V.

C.—Heterostatic Action.

When $m = n = 1$ gives the smallest values of Q_a and Q_b in Formula (34), Formula (37) may be used for the moment at the center per unit width:

$$M'_{\max.} = 0.077\ w\ b^2 \left(1 + 1.07 \frac{P_a}{Q_a - P_a}\right) \quad \ldots\ldots\ldots (37)$$

(to be compared with Formula (32) applying to the homogeneous slab).

IV.—General Theory of Structural Actions in which the Stresses are not Proportional to the Loads, although Hooke's Law Applies in Each Element of the Structure, and although the Deformations Are Small.

12.—The Energy Equation, the Loads, and the Parameters.—In the Introduction, Article 1, definitions were given of three general types of elastic action: Orthostatic or regular action, in which the stresses are proportional to the loads; astatic action in which pure buckling occurs; and heterostatic or mixed action. In the two latter types of action, the stresses are not, in general, proportional to the load. The two assumptions common to all cases considered are recalled, namely, that Hooke's law of proportionality of stresses and strains applies in each structural element, and that the deformations are small. It is now proposed to investigate some of the general features of astatic and heterostatic actions. The energy principle, or the principle of least action, is found to be particularly effective for this purpose. This principle makes it possible to express the conditions of equilibrium by one condition or equation — the energy equation. The following notation will be used:

Let U = total internal elastic energy stored in the structure by the deformation, or, the elastic potential energy of the structure. This quantity is called by many writers the resilience* of the structure, and this term will be used in this paper.

P = external load in the generalized sense, or "generalized force". P is a quantity which may represent one force or a group of forces acting on the structure. When all the forces in this group are increased by a certain proportion, this increase is represented by an increase of P by the same proportion. P, therefore, is a quantity measuring the intensity of load in the whole group of forces. The load, P, is the "generalized force" which was introduced by Lagrange in his general equations of dynamics.

δ = displacement in the direction of the load, P. If P is a single force, δ is the component of the linear displacement of the point where P is applied, in the direction of P. If P represents a group of forces, δ is such a quantity that $P\delta$ is the work done by the load, P, while the elastic displacements increase from zero to their final values, P being constant during this process. For example, if P is a torque (couple), δ is the angular rotation (in radians) of the structural part on which the couple is acting. If the load, P, consists of two forces each equal to P, acting against one another at the end points of a certain

* In Johnson's "Materials of Construction", 1919 edition, p. 38, resilience is defined as "the work which a body can do in springing back after a deforming force has been removed". The present use of the term is in accordance with this definition. See, also, A. E. H. Love, "Mathematical Theory of Elasticity", 1906 edition, p. 170, and J. A. Ewing, "Strength of Materials", 1906 edition, p. 15. There has been some disagreement as to the use of the term, resilience; for example, E. S. Andrews ("Strength of Materials", 1915, pp. 33, 272) uses the term in the sense of energy per unit volume.

distance, δ is the shortening of this distance. If the load, P, consists of two forces each equal to $\frac{1}{2}P$, acting against one another at the end points of a certain distance, δ is one-half the shortening of the distance, because only then could $P\delta$ be equal to the work of the load. If the load, P, consists of the individual forces, $P_1, P_2, \ldots\ldots P_n \ldots\ldots$, in the direction of which the displacements are, $\delta_1, \delta_2, \ldots\ldots \delta_n \ldots\ldots$, δ is found from the equation, $P\delta = \Sigma P_n \delta_n$. In Lagrange's dynamics, δ is the generalized displacement, or the change of a generalized co-ordinate.

$L =$ the difference between the internal energy or resilience of the structure and the work which is done by the external forces during the deformation, if the external forces remain constant during this process. That is, when P is the only load, $L = U - P\delta$. L may then be considered as the total potential energy of the structure and loads.

$u_1, u_2 \ldots\ldots u_m \ldots\ldots u_n \ldots\ldots =$ parameters of the structure which define the elastic displacements, or, in terms of Lagrange's dynamics, "generalized co-ordinates" of the structure. Unless otherwise stated, zero displacements correspond to $u_1 = u_2 \ldots\ldots = u_m \ldots\ldots u_n \ldots\ldots = 0$. Definite values assigned to each of the parameters will define completely the elastic changes of shape, that is, the displacements are functions of the parameters, u. In most cases, the number of parameters is infinite. In the case of an originally straight, incompressible, but flexible column, the constants in the Fourier series, expressing the deflections in terms of the distance along the column, may be used as parameters (compare Article 8) and they constitute a complete set. U and δ are functions of the parameters.

The general condition of equilibrium when P is the only load, may now be written:

$$L = U - P\delta = \pm \text{ min.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

The $\pm$ min. stands for "minimum or maximum or condition of zero derivatives". A minimum of L corresponds to a stable equilibrium, a maximum to an unstable equilibrium. As the cases of stable equilibrium, giving a minimum, are the more important—in orthostatic action this case will be found to be the only one possible—and as the definite conditions derived by differentiation of the equation are the same whether a maximum or a minimum occurs, the condition of equilibrium in the standard form may be written:

$$L = U - P\delta = \text{min.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

where the "minimum" on the right side is understood as a $\pm$ min., namely, usually a minimum, but, in special cases, a maximum. When the minimum occurs, Formula (39) expresses the "principle of least action". By differ-

entiating Formula (38) or Formula (39), with respect to the parameters, it will be found that:

$$\cdots \frac{\partial L}{\partial u_m} = 0 \cdots\cdots \frac{\partial L}{\partial u_n} = 0 \cdots\cdots\cdots\cdots\cdots\cdots (40)$$

giving one equation for each parameter. The set of equations, Formula (40), is a complete equivalent of the energy equation, Formula (38) or Formula (39).

13.—The Internal Actions or Generalized Stresses, and the Elastic Potential Energy or Resilience.—The stresses at any one point measure the intensity of the internal elastic action. The stresses, however, are not the only quantities which can be used for this purpose; for example, in a beam which is bent by transverse forces only, the bending moment, M, is a measure of the intensity of action in a longitudinal element of length, ds, enclosed between two consecutive cross-sections. When EI is the product of the modulus of elasticity and the moment of inertia of the cross-section, $\frac{M^2\,ds}{2\,E\,I}$ is the internal energy or resilience of the element. If, in addition, there is an axial force, P, this force gives an additional stress uniformly distributed over the area, A, of the cross-section, and creates in the same structural element an additional element of energy equal to $\frac{P^2\,ds}{2\,E\,A}$. Therefore, P is also a "quantity measuring internal action". Usually, in beams and columns, the influence of the shearing stresses on the deformations is negligible. That is, the bending moments, M, together with the axial load, P, determine completely the internal energy or resilience of the column. M and P, therefore, may be called stresses in the generalized sense. In the general case, the action of a structure may be described in terms of some set of "internal actions" or "generalized stresses", R, which have such properties that the element of energy is written $\frac{1}{2}\,r\,R^2\,ds$, in which r is a function of the location and s measures "extension of the structure". The total elastic potential energy or the resilience is then expressed:

$$U = \frac{1}{2}\int r\,R^2\,ds \cdots\cdots\cdots\cdots\cdots\cdots (41)$$

where the integration is to be extended over the whole structure, and is to include all the elements of energy. It is always possible to indicate quantities such as R, which give an energy expression in the Formula (41).* As in the

*Let dV = volume element of a homogeneous isotropic elastic solid;
X, Y, Z = normal stresses in the directions, x, y, z;
S_{yz}, S_{zx}, S_{xy} = the corresponding shearing stresses;
E = modulus of elasticity;
K = Poisson's ratio of lateral contraction to longitudinal elongation.
Then, the elastic potential energy of the solid may be expressed:

$$U = \frac{1}{2}\int \left[\frac{1}{E}(X^2 + Y^2 + Z^2) - \frac{2K}{E}(YZ + ZX + XY) + \frac{2(1+K)}{E}(S^2_{yz} + S^2_{zx} + S^2_{xy})\right] dV \cdots\cdots\cdots (41a)$$

case of the axially compressed beam, the same element, ds, may be included more than once in the integral of the resilience.

It will be assumed that the actions, R, are linear functions, and, unless otherwise stated, linear homogeneous functions of the parameters, $u_1, u_2 \ldots\ldots$ When only small deformations of the structure are considered, it is always possible to choose a set of parameters satisfying this condition; and when one set of such parameters has been found, this set may be replaced by any other set in which each parameter is a linear homogeneous function of the parameters of the first set. Formula (42) may then be written:

$$R = R_1 u_1 + R_2 u_2 + \ldots\ldots + R_m u_m \ldots\ldots + R_n u_n + \ldots\ldots \quad (42)$$

where $R_1 \ldots\ldots R_n \ldots\ldots$, are functions of the location, that is, functions of s. Substituting Formula (42) in Formula (41) the expression for the internal energy or resilience is found:

$$U = \left(\frac{1}{2}\right) \sum_m u^2_m \int r R^2_m \, ds + \sum\sum_{mn} u_m u_n \int r R_m R_n \, ds \ldots\ldots \quad (43)$$

where $m = 1, 2 \ldots\ldots, n = 1, 2, \ldots\ldots$, and $m \neq n$, and where the integrations are extended so as to include all elements of energy. The integrals in the summations may be considered as "constants of the structure", and will be denoted as follows:

$$U_m = \int r R^2_m \, ds; \; U_{mn} = \int r R_m R_n \, ds \ldots\ldots\ldots\ldots \quad (44)$$

The resilience is then expressed:

$$U = \frac{1}{2} \Sigma U_m u^2_m + \Sigma \Sigma U_{mn} u_m u_n \ldots\ldots\ldots\ldots\ldots\ldots \quad (45)$$

where $m \neq n$. It is noted that U is a quadric in the parameters (a quadric is defined as a homogeneous function of second degree).

The additional notation introduced is summarized here as follows:

Let R = generalized stress, or, quantity measuring internal action;

ds = dimension of the structural element;

r = function of the location, depending on the stiffness of the element in such a way that $\frac{1}{2} r R^2 \, ds$ is equal to the corresponding element of energy or resilience;

As Formula (41a) contains the products, YZ, ZX, and XY, the stresses X, Y, and Z cannot be used as R's in Formula (41), except when two of these stresses are zero. Formula (41a), however, may be written in the form, Formula (41), for example, as follows:

$$U = \frac{1}{2} \int \left[\frac{1-2K}{3E} (X+Y+Z)^2 + \frac{1+K}{2E} (X-Z)^2 + \frac{1+K}{6E} (X-2Y+Z)^2 + \frac{2(1+K)}{E} (S^2_{yz} + S^2_{zx} + S^2_{xy}) \right] dV = \frac{1}{2} \int (\Sigma r R^2) \, ds \ldots\ldots \quad (41b)$$

That is, Formula (41) may be used with the following set of generalized stresses, R, corresponding to the volume element, dV or ds:

$$R^{\mathrm{I}} = X + Y + Z; \; R^{\mathrm{II}} = X - Z; \; R^{\mathrm{III}} = X - 2Y + Z; \; R^{\mathrm{IV}} = S_{yz}; \; R^{\mathrm{V}} = S_{zx}; \; R^{\mathrm{VI}} = S_{xy} \ldots\ldots \quad (41c)$$

$R_1, R_2 \ldots\ldots R_m \ldots\ldots$ = coefficients of the parameters in the linear expression of R. These coefficients are functions of the location;

$\ldots\ldots U_m \ldots\ldots U_{mn} \ldots$ = constants of the structure, which are coefficients in the quadric expressing the resilience.

14.—The Case of Orthostatic Action.—The energy equation, Formula (39), and its equivalent, Formula (40), give equations of the form:

$$\frac{\partial L}{\partial u_m} = \frac{\partial U}{\partial u_m} - P\frac{\partial \delta}{\partial u_m} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

namely, one such equation for each parameter. Since U is a quadric in the parameters (Formula (45)), the derivatives, $\frac{\partial U}{\partial u_m}$, are linear homogeneous functions of the parameters. In orthostatic action, the stresses, and, therefore, the parameters, are linear functions of the load, hence, if P is to give orthostatic action, the derivatives, $\frac{\partial \delta}{\partial u_m}$, must be independent of the u's; that is, they must be constants; or, δ in orthostatic action is a linear function of the parameters. It is assumed, in accordance with the general rule which was adopted, that the state of zero stresses and zero deformations is represented by zero values of the parameters. Then δ is a linear homogeneous function of the parameters.

*15.—Astatic Action, Astatic Parameters, and the Theorem of Orthogonality.**—It is now assumed that P is a load which can produce astatic action. Let Q_m be a value of P at which buckling is possible; that is, Q_m is a "critical value" of P. When P becomes equal to Q_m, then the structure enters into a state of astatic elastic equilibrium, that is, an equilibrium maintained at constant load throughout a continuous range of configurations of the structure. The parameter, u_m, will be chosen in a manner which correlates it peculiarly with the buckling under the load, Q_m. Three configurations or shapes of the structure will be considered, which will be called Configurations Nos. 1, 2, and 3. Configurations Nos. 1 and 2 are selected among the shapes of the structure which occur in the process of buckling under the load, Q_m. The change of the structure from Shape No. 1 to Shape No. 2 will be represented by a certain set of changes in the stresses or generalized stresses. The change of each generalized stress, R, will be expressed as the product, $R_m u_m$, of a fixed chosen value, u_m, times a factor, R_m. One such factor, R_m, belongs to each stress which is represented. The whole set of changes of stresses is defined completely when the set of such factors, R_m, and the quantity, u_m, are given. Now, assume that the value of u_m is changed, and that each factor, R_m, is multiplied by the new value of u_m. The products define a set of changes of the stresses. When the changes are measured from the initial configuration, No. 1, the set of changes defines a possible shape of the structure. This shape will be recorded as Configuration No. 3.

* It might seem that a simpler definition of the "astatic parameter" than the one given here could be found. The apparent complication is due to the fact that the definition is to apply, not only to the simplest cases, but also to the cases described later as "double astatic equilibrium" (Article 16) and "mixed astatic action" (Article 23).

The changes of stresses (generalized stresses or internal actions) caused by the transformation from Shape No. 1 to Shape No. 3 are proportional to the corresponding changes which occur when the structure is transformed from Shape No. 1 to Shape No. 2. The ratio between the two sets of changes is the ratio of the corresponding values of u_m. The same ratio exists between the corresponding changes of internal actions in general, and of other quantities which, like the deflections in general, are linear functions of the stresses. In other words, when u_m is used as a common factor defining changes of stresses, the variation of u_m alone will define a range of variations of shape; or u_m, used as indicated, serves as a parameter of the structure. Introducing u_m as such, and summarizing the results, we have: The changes through Configurations Nos. 1, 2, and 3 may be defined by the variation of the parameter, u_m, while the remainder of the parameters retain the constant values which they have in Configuration No. 1. Configuration No. 1, the initial shape from which the changes are measured, is represented by $u_m = 0$.

The general equation of equilibrium, Formula (39), is again written:

$$L = U - P\delta = \min.$$

U is a quadric in the parameters. δ is some function of the parameters, but not a linear function, because, in accordance with Article 14, a linear function would lead to orthostatic action. As $P = Q_m$ gives equilibrium throughout a continuous range of variations, it follows that δ must be a function which resembles U in some respects. It might be assumed, then, that δ is a quadratic function of the parameters, or, more exactly, that in expressing δ as a polynomial or power series, terms of higher than second degree in the parameters can be thrown away as negligible. In some cases, one may verify by inspection of the structure the statement that δ is such a function.

By the following reasoning, one may conclude that, in the general case, δ is expressed with sufficient approximation as such a quadratic function of the parameters. The function, δ, is an elastic displacement. By the assumption of rigidity made in the Introduction, the displacements are small, and they may be considered as infinitesimal quantities when compared with the main dimensions of the structure, or, the displacements may be considered as small increments to the dimensions. They can be expressed in terms of the parameters by Taylor expansions. As the increments are small, the prevailing condition, special displacements being excepted, will be that in these expansions the terms of second and higher degree are negligible when compared with the linear terms. Such displacements, therefore, are expressed with sufficient approximation as linear functions of the parameters. In the special, exceptional case of the displacement, δ, in the direction of an astatic load, P, and in some related cases, it is not possible to throw away all terms of higher than first degree in the parameters. It follows from the usual properties of expansions in the Taylor series that, in such cases, it can be expected, and it will be true in general, that a sufficient approximation is obtained by including the terms of second degree in the parameters, neglecting terms of higher degrees. On this ground, it may be assumed that δ in Formula (39), in the case of astatic action, can be expressed sufficiently exactly as a quadratic

function of the parameters. In addition, however, as will be shown later, a criterion may be found for the correctness of this assumption in the very fact that astatic equilibria are known to exist; this knowledge may have been derived from other analyses or from tests.

When both U and δ are quadratic functions of the parameters, it follows that the formulas, derived by differentiating the energy equation, Formula (39), are linear in the parameters. That is, Formulas (40),

$$\ldots\ldots \frac{\partial L}{\partial u_m} = 0 \ldots\ldots \qquad \frac{\partial L}{\partial u_n} = 0 \ldots\ldots\ldots$$

are linear in $\ldots u_m \ldots u_n \ldots$ also in this case of astatic action. When $P = Q_m$, the set of Formulas (40) is satisfied by the sets of values of the parameters which correspond to Configurations Nos. 1 and 2, both of which belong to the astatic equilibrium under the load, Q_m. That is, the set of Formulas (40) is satisfied by two different values of u_m, namely, $u_m = 0$, corresponding to Configuration No. 1, and u_m equal to the first chosen finite value, corresponding to Configuration No. 2. The remainder of the parameters are the same in the two cases. It follows that, in Formulas (40), all coefficients to u_m vanish when $P = Q_m$. Formulas (40), however, will be satisfied by all values of u_m as long as the remainder of the parameters are as shown in Configuration No. 1. That is, the whole range of variations of shape—defined by u_m equals any value, all other parameters being equal to definite values, namely, their values in Configuration No. 1—represents a state of astatic equilibrium produced under the critical load, Q_m. This range contains as possible cases Configurations Nos. 1, 2, and 3.

We shall call u_m an astatic parameter. An astatic parameter is, therefore, by definition, a parameter which by its own variation, while the remainder of the parameters remain constant, defines the various stages of a state of astatic equilibrium.

As an example, take the case of a hinged-end column, of length, l, with constant cross-section, with a stiffness factor, EI, and with axial loads at the ends only. At the critical end load, $Q_m = \dfrac{\pi^2 m^2 EI}{l^2}$, an astatic equilibrium exists in which the deflected curve consists of m half waves. When x equals the distance from one end, the deflections in this buckling can be expressed as $y = u_m \sin \dfrac{m\pi x}{l}$, in which u_m may take any value. A change of u_m in this expression defines proportional changes of all stresses, curvatures, and transverse deflections. It follows that u_m, or any quantity proportional to it, can be used as an astatic parameter. In the general case of flexure, under the influence of an end load combined with transverse loads, the deflections can be expressed as $y = \Sigma u_n \sin \left(\dfrac{n\pi x}{l}\right)$, where $n = 1, 2 \ldots\ldots m \ldots\ldots$, etc. Evidently, the whole set of coefficients, $u_1 \ldots u_m$, can be used as parameters, and they will all be astatic parameters. By introducing in addition to $u_1 \ldots u_m \ldots$, a parameter which measures the axial shortening of the column, one obtains a complete set of parameters in terms of which any

possible variation of shape can be described. It follows that the astatic equilibrium under the load, Q_m, is represented by u_m, being indefinite, equal to any value, other parameters being equal to definite values, namely, some finite value for the parameter measuring the axial shortening, and the remainder being equal to zero.

The general case is now considered again. The energy Formula (39):

$$L = U - P\delta = \text{min.}$$

will be used. It has been shown that when δ is known to be quadratic, two configurations, belonging to an astatic equilibrium, determine a corresponding astatic parameter. In the discussion which follows, we may abandon temporarily the assumption that δ is known beforehand to be a function of not higher than second degree in the paramcters, even though this assumption is known to represent the general case. It will be assumed only that an astatic equilibrium is known to exist, and that a corresponding parameter, u_m, exists, which is an astatic parameter in the sense of the definition just given; that is, the variation of u_m, while the rest of the parameters remain constant, represents an astatic equilibrium produced under a critical load, say, $P = Q_m$. In this astatic equilibrium, it is found, as usual, that all the first partial derivatives of L with respect to the parameters must vanish. Moreover, since u_m can vary through a continuous range of values without disturbing the condition of minimum, it follows that, also, all second and higher derivatives of L in which differentiation with respect to the astatic parameter, u_m, occurs are zero. Hence, if u_n is any other parameter, we have, when $P = Q_m$:

$$\frac{\partial^2 L}{\partial u_m^2} = 0, \qquad \frac{\partial^2 L}{\partial u_m \partial u_n} = 0, \ldots\ldots\ldots\ldots\ldots (47)$$

Now, assume that also u_n is an astatic parameter, and that its corresponding critical astatic load is Q_n; that is, u_n equals any value, other parameters being equal to definite values, represents an astatic equilibrium in which $P = Q_n$ is the load. Similar to Formula (47), we have then, when $P = Q_n$:

$$\frac{\partial^2 L}{\partial u_n^2} = 0, \qquad \frac{\partial^2 L}{\partial u_m \partial u_n} = 0, \ldots\ldots\ldots\ldots\ldots (48)$$

The resilience quadric may be written in the form:

$$U = \frac{1}{2} U_m u_m^2 + \frac{1}{2} U_n u_n^2 + U_{mn} u_m u_n + \bar{U} \ldots\ldots\ldots (49)$$

where U_m, U_n, and U_{mn} are constants and where $\bar{U}$ is a function which has no terms containing u_m^2, u_n^2, or $u_m u_n$. It follows, from the relative position of U and δ in the energy equation and from the fact that all the partial derivatives of L with respect to u_m and u_n must vanish, that δ may be written in the form:

$$\delta = \frac{1}{2} \delta_m u_m^2 + \frac{1}{2} \delta_n u_n^2 + \delta_{mn} u_m u_n + \delta \ldots\ldots\ldots (50)$$

where δ_m, δ_n, and δ_{mn} are constants, and where δ is a function in which terms containing u_m^2, u_n^2, $u_m u_n$, or products of higher degree in u_m and u_n, do not exist.

Substituting Formulas (49) and (50) in the expression, $L = U - P\delta$, and differentiating in accordance with Formula (47), we find:

$$U_m - Q_m \delta_m = 0 \quad \ldots\ldots(51)$$

$$U_{mn} - Q_m \delta_{mn} = 0 \quad \ldots\ldots(52)$$

In the same manner, differentiating according to Formula (48), we find:

$$U_n - Q_n \delta_n = 0 \quad \ldots\ldots(53)$$

$$U_{mn} - Q_n \delta_{mn} = 0 \quad \ldots\ldots(54)$$

Formulas (51) and (53) may be written:

$$Q_m = \frac{U_m}{\delta_m}\ ;\ Q_n = \frac{U_n}{\delta_n} \quad \ldots\ldots(55)$$

These simple formulas determine the astatic loads in terms of the coefficients in the functions, U and δ.

It is assumed at present that the two critical astatic loads, Q_m and Q_n, are different. It follows that Formulas (52) and (54) can be satisfied only when

$$U_{mn} = \delta_{mn} = 0 \quad \ldots\ldots(56)$$

that is, the mixed terms in U and δ containing the product, $u_m\ u_n$, vanish when the two parameters correspond to different critical astatic loads.

Since by Formula (44),

$$U_{mn} = \int r R_m R_n\, ds$$

we have,

$$\int r R_m R_n ds = 0 \quad \ldots\ldots(57)$$

The vanishing of this integral represents an important property of the functions, R_m and R_n, namely, the property of orthogonality. Two functions, $\phi\ (x)$ and $\psi\ (x)$, are called orthogonal in a certain field when their product integrated over the field vanishes, that is, when $\int \phi\ (x)\ \psi\ (x)\ dx = 0$. If R_m is taken as the one function, R_n, as the other, and $rds = dx$, it is seen that Formula (57) expresses this property. When there is a whole set of orthogonal functions—as will usually be the case in astatic action—these functions become useful by their adaptability to expansions in series, particularly in problems relating to the structures in which the functions are generated. This result will be brought out by the theory of heterostatic action which follows later. The theorem expressed by Formula (57) will be referred to as the theorem of orthogonality.*

As an example, consider the case of an hinged-end column, of the length, l, with a constant stiffness factor, EI, and with $s = x =$ distance from one end. The bending moments will be used as a measure of internal action, R, that is, as

* Sets of orthogonal functions play an important part in the theory of integral equations, and, therefore, it would be expected that the subject treated here would furnish an example of application of that theory. As a matter of fact, various elements in the subject of integral equations will appear to be represented directly and physically in the problems treated. For instance, the sets of characteristic numbers, or auto-values, in the homogeneous integral equations are represented as proportional to the sets of astatic loads. Concerning the theory of integral equations, see, for example, E. T. Whittaker and G. N. Watson, "Modern Analysis", Second Edition, 1915, pp. 205 *et seq.*

generalized stresses. The bending moments in two different astatic actions may be expressed as:

$$M_m = C_m \sin \frac{m\pi x}{l}, \text{ and } M_n = C_n \sin \frac{n\pi x}{l}$$

where $m \neq n$. It is well known that with $m \neq n$,

$$\int_0^l \sin \frac{m\pi x}{l} \times \sin \frac{n\pi x}{l} \times dx = 0$$

that is, with $R_m = M_m$, $R_n = M_n$, and $r = \frac{1}{EI}$, Formula (57) is satisfied.

Formula (50) shows that the part of δ which depends on the astatic parameters, u_m and u_n, alone is a quadratic function. This property is seen readily to extend to any number of astatic parameters. If in some manner or other certain parameters are known to be astatic, a corresponding part of δ will exist, which is quadratic in these parameters. Herein, is the criterion referred to previously by which, in many cases, the character of δ may be verified as a function of the parameters not higher than quadratic. This statement applies particularly to a group of cases, which will be shown to be important, namely, a group in which δ can be expressed in terms of astatic parameters, together with only one other parameter. The latter will be shown to enter into δ as a linear term; or, δ is in such cases a quadratic function of the set of parameters.

16.—Double Astatic Equilibrium.—In Article 15, the assumption was made that the two astatic loads, Q_m and Q_n, are different. The case will now be considered in which $Q_m = Q_n = Q_{mn}$. Formulas (51) to (54) are satisfied in this case when

$$Q_{mn} = \frac{U_m}{\delta_m} = \frac{U_n}{\delta_n} = \frac{U_{mn}}{\delta_{mn}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (58)$$

That is, in this special case, the mixed terms, containing u_m u_n, and having U_{mn} and δ_{mn} as coefficients, do not necessarily vanish, and the property of orthogonality is not necessarily found. Another result is that any combination of values of the two parameters, u_m and u_n, with the other parameters remaining constant, represents a possible equilibrium; that is, a double neutral or double astatic equilibrium occurs.

Some examples will illustrate this case: Consider a long cylindrical vertical column with such spherical bearings at the ends as to allow the ends to slope freely in any direction. At the lowest critical load given by the Euler formula, the column may buckle in any vertical plane through the original center line. Any such buckling may be found by superimposing a buckling curve in, say, the north-south plane on one in the east-west plane. Inversely, superposition of a buckling curve in the north-south plane on a buckling curve in the east-west plane, produced under the same critical load, leads to a possible buckling curve which will be contained in some vertical plane through the original center line. The equilibrium is a typical two-parametric astatic equilibrium, with two parameters assuming any values, while the remainder of the parameters have definite values. Another example is the thin-shelled circular cylinder carrying an outside pressure (Fig. 1 (*G*)). If the thickness and the material are uniform, the surface may buckle at the critical loads into

a wave line starting at any point. As both the maximum deflection and the starting point can vary, we have again a two-parametric astatic equilibrium. The complete set of buckling surfaces at that load can be found by superimposing the buckling waves starting at two definite points. A third example is the case of the rectangular slab carrying compressive loads along its edges. It was mentioned in Article 10 that, at certain combinations of the end load, the slab may buckle into two different types of waved surfaces and into any surface found by superposition of waves of these two types.

It is evident that triple or higher-multiple astatic equilibria may also be found. In the case of an hinged-end circular column with horizontal spring supports at the ends (compare Fig. 1 (*F*)), the springs may yield and the column may buckle in two different planes at the same critical load. This would be a triple astatic equilibrium.

17.—General Formulas in Simplified Case.—Assume that astatic action can be produced under the critical astatic loads, Q_1, $Q_2 \ldots\ldots Q_n \ldots\ldots$, and let u_1, $u_2 \ldots\ldots u_n \ldots\ldots$ be the corresponding set of astatic parameters, so that, for instance, the combination, u_n = any value, other u's = 0, represents the astatic action produced when $P = Q_n$. The simplifying assumptions are made that this set of parameters is sufficient to define all the possible shapes of the structure, and that no two parameters correspond to the same Q; that is, all the Q's are different. For instance, if the axial shortenings can be neglected as being only of small influence, then the action of a slender straight column, flexible in one plane only, is described completely by an equation for deflections, of the form

$$y = \Sigma\, u_n \sin \frac{n\,\pi\,x}{l}$$

where x is the distance from one end. If the ends are hinged, and the stiffness factor, EI, is constant, then the u's in this expression are astatic parameters, and the combination, u_n = any value, other u's = 0, corresponds to the critical astatic load,

$$Q_n = \frac{n^2\,\pi^2\,E\,I}{l^2}$$

In the general case, which will now be considered, the load is assumed to consist of two component parts which may vary independently: One part is denoted by P, and is an astatic load, which, acting alone, produces astatic equilibrium when it assumes any one of the critical values, $Q_1 \ldots\ldots Q_n \ldots$; the other part, W, is an orthostatic load, producing orthostatic action when acting alone. W cannot be a part of P. The combination of the two components, P and W, is a heterostatic load of a general type, producing heterostatic action. As before, δ is the displacement in the direction of P. In a similar manner, w will denote the displacement in the direction of the load, W, that is, Ww is the work of the load, W. The first object will be to find the parameters in the various possible cases. For the purpose of distinguishing between the cases, the particular values of the parameters occurring in the heterostatic action will be indicated by marking them with a prime, so that u'_n is the value of the parameter, u_n, in the heterostatic action. Without the prime,

u_n either represents the parameter as a variable quantity, or, when considered as a definite quantity, it denotes the particular value of the parameter occurring in the orthostatic action which is produced by W alone. When the parameters have been found, the various effects, such as deflections, bending moments, etc., that is, all generalized stresses, generalized strains, and generalized displacements, can be determined. These effects will be assumed to be linear homogeneous functions of the parameters. The "effects" will be denoted by F and F'. F will represent either "the effect in general", as a function of the parameters, or the particular value of the effect due to the orthostatic load, W, alone, while F' is the particular value of the effect in the heterostatic action. It is seen that the use of the prime is the same here as in the case of u_n and u_n'. Similarly, R' will denote the internal action in the heterostatic action, while R denotes either the internal action in general or the particular value of the internal action which occurs in the orthostatic action.

The notation introduced here is as follows:

Let
L = combined potential energy of structure and loads.
L = 0 when the displacement is zero.
U = resilience of the structure.
P = astatic component of the load.
W = orthostatic component of the load.
δ = displacement in the direction of the load, W, defined by Ww = work of W.
$Q_1, Q_2 \ldots\ldots Q_n \ldots\ldots$ = critical values of the astatic load, P.
$u_1 \ldots\ldots u_n \ldots\ldots$ = set of astatic parameters corresponding to the astatic load, P; in particular, when they are considered as definite quantities, they are the values of the parameters in the orthostatic action.
$u'_1 \ldots\ldots u'_n \ldots\ldots$ = same parameters in the heterostatic action.
R = internal action, particularly in the orthostatic action.
R' = internal action in the heterostatic action.
ds = extension of structural element.
r = stiffness constant, so defined that $\frac{1}{2}\, rR^2 ds$ is an element of the resilience.
F = effect depending on the parameters, particularly the effect in the orthostatic action. F is a linear homogeneous function of the parameters.
F' = same effect in the heterostatic action.

The writer will show first how the parameters may be found in any given case, provided the internal actions are known. The intensity of internal action in any one element was expressed by Formula (42) as:

$$R = R_1 u_1 + \ldots\ldots R_m u_m + \ldots\ldots R_n u_n + \ldots\ldots$$

where $R_m \ldots R_n$ are functions of the location. u_n = any value, other u's being equal to zero, represents the astatic action produced by $P = Q_n$. Therefore, $R_n u_n$ is the internal action in the astatic action, Q_n, and the function,

R_n, has the particular significance of being the internal action in the astatic action, Q_n, when $u_n = 1$. For example, the bending moment, M, in Formula (11), which has reference to the general case of straight columns, may be chosen as a measure of the internal action, that is, in this case, $R = M$. The function, M_n, in Formulas (11) and (12), was defined as the bending moment in a possible buckling action, that is, we may write $R_n u_n = M_n$. Thus, Formula (11) appears as a special type of Formula (42). The general case is now considered again. Multiply Formula (42) by $rR_n ds$ and integrate so as to include all elements of energy. On account of the theorem of orthogonality, which states that $\int r\, R_m\, R_n\, d\, s = 0$, when $m \neq n$, all terms on the right side with an index different from n will vanish by the integration, and we find,

$$\int r\, R\, R_n\ d\, s = u_n \int r\, R_n^{\,2}\, d\, s$$

or,

$$u_n = \frac{\int r\, R\, R_n\, d\, s}{\int r\, R_n^{\,2}\, d\, s} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (59)$$

The simplicity of Formula (59) shows the particular advantage of the sets of orthogonal functions generated in the astatic elastic actions. When the parameters have been found, their values may be substituted in the expressions for the various effects in the structure. Such effects are represented in general by a series of the form:

$$F = F_1\, u_1 + \ldots\ldots + F_m u_m + \ldots\ldots + F_n u_n \ldots\ldots\ldots (60)$$

where $F_m \ldots\ldots F_n$ are functions depending on the location and on the type of the effect. The internal action, R, represents a special case of the general effect, F.

The energy equation will now be discussed. In the general case, in which both loads, P and W, are acting, the energy equation takes the form:

$$L = U - P\delta - Ww = \min. \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (61)$$

the minimum here, as in Formula (39), being understood as a $\pm$ minimum. $P = 0$ gives orthostatic action, while $W = 0$ gives astatic action.

As all the critical loads, Q, are different, the quadric, U, does not contain any products of the parameters (compare Article 15), therefore, U may be written as a series,

$$U = \frac{1}{2} \Sigma\, U_n\, u_n^{\,2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (62)$$

where the coefficients, U_n, are constants. It also follows that the quadratic function, δ, cannot contain products of the parameters. It will be shown a little later that δ does not contain any linear terms. δ is then a quadric similar to U and may be written:

$$\delta = \frac{1}{2} \Sigma\, \delta_n\, u_n^{\,2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (63)$$

where the coefficients, δ_n, are constants. It was shown in Article 14 that the displacements in the direction of orthostatic loads are linear homogeneous functions of the parameters, therefore, w may be written:

$$w = \Sigma\, w_n u_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (64)$$

where $\ldots\ldots w_n \ldots\ldots$ are constants.

The three actions, astatic, orthostatic, and heterostatic, will now be considered separately.

A.—Astatic Action.

In this case, $W = 0$. Then the energy equation takes the form:

$$L = U - P\delta = \text{min.}$$

The state of non-buckling always represents a possible equilibrium whatever the load, P, is. Hence, $u_1 = \ldots\ldots = u_n \ldots\ldots = 0$ is one possible solution of the energy equation. Linear terms in the parameters contained in δ would make some of the derivatives, $\frac{\partial \delta}{\partial u_n}$, contain constant terms which would prevent the vanishing of the corresponding derivatives, $\frac{\partial L}{\partial u_n}$, at $u_1 = \ldots\ldots u_n = \ldots\ldots = 0$. Therefore, the proposition is correct, which was mentioned previously, namely, that there cannot be any linear terms in the expression for δ. Therefore, Formula (63) may be used. When $P = Q_n$, the condition, $\frac{\partial^2 L}{\partial u_n^2} = 0$, leads again to Formula (53), which may be written as:

$$U_n = Q_n\, \delta_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (65)$$

When this relation is satisfied, the combination $P = Q_n$, $u_n =$ any value, all other u's $= 0$, makes all the first derivatives of L, with respect to the parameters, vanish, and, therefore, this combination represents a correct solution of the energy equation.

Formula (65) allows one to express the resilience quadric in the form:

$$U = \frac{1}{2}\, \Sigma\, Q_n \delta_n u_n^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (66)$$

B.—Orthostatic Action.

When $P = 0$, the energy equation becomes:

$$L = U - Ww = \text{min.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (67)$$

By substituting Formulas (62) and (64), and writing $\frac{\partial L}{\partial u_n} = 0$, we find:

$$U_n u_n - W w_n = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (68)$$

whereby the parameters may be determined when the coefficients in the expressions for U and w are known. On account of Formula (65), Formula (68) may also be written:

$$Q_n \delta_n u_n - W w_n = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (69)$$

Formulas (68) and (69) show that the parameters, u_n, are proportional to the load, W.

In a number of cases of orthostatic action, the parameters are found most conveniently by Formula (59). Since by Formula (44), $U_n = \int r\ R_n^2\ ds$, Formula (68) becomes, in fact, the same as Formula (59). The expression for w_n follows:

$$w_n = \int r\ R\ R_n\ d\ s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (70)$$

Examples of the use of these methods in the analysis of orthostatic action may be found in Article 8. A part of the total resilience in the columns treated in Article 8, is due, in general, to the axial compression of the column, and this part would depend on some parameter other than the astatic parameters. A phase of the problem is involved here, which will be studied more closely in some of the subsequent articles. It will then be shown that such effects as the axial shortening do not change the applicability of the formulas and methods, just given, to the cases examined in Article 8. In the first case treated, that of hinged-end, straight columns, M_n in Formula (13) takes the place of $R_n u_n$. By choosing the coefficient, C_n, in Formula (13) as the parameter, u_n, it is found that $R_n = \sin \frac{n\ \pi\ x}{l}$. In this case, the factor, $r = \frac{1}{E\ I}$, is a constant which cancels when substituted in Formula (59). With $d\ s = d\ x$, it is found that,

$$\int R_n^2\ d\ s = \int_0^l \sin^2 \frac{n\ \pi\ x}{l}\ d\ x = \frac{l}{2}$$

The substitutions, $R = M$; $u_n = C_n$, transform Formula (59) into Formula (14), or

$$C_n = \frac{2}{l} \int_0^l M \sin \frac{n\ \pi\ x}{l}\ d\ x$$

Formula (14), therefore, may be interpreted as a special case of the general Formula (59). Integration of Formula (14) led to the series for M given shortly after Formula (14). This series exemplifies the expressions for R and F (Formulas (42) and (60)). Also, the other cases treated in Article 8 may be considered as examples of the more general theory.

C.—Heterostatic Action.

Both P and W are assumed to be different from zero. The energy equation, therefore, takes its general form, Formula (61). When U is taken from Formula (66), δ from Formula (63), and when the parameters are marked with a prime, the energy equation becomes:

$$L = \frac{1}{2} \Sigma\ (Q_n - P)\ \delta_n\ u'^2 - W\ \Sigma\ w_n\ u'_n = \min \ldots\ldots\ldots (71)$$

Differentiation with respect to the parameter, u'_n, gives:

$$\frac{\delta\ L}{\delta\ u'_n} = (Q_n - P)\ \delta_n\ u'_n - W\ w_n = 0 \ldots\ldots\ldots\ldots (72)$$

A comparison of Formula (72) with Formula (69) leads to the following expression for the parameter, u'_n, in the heterostatic action, in terms of the

value, u_n, of the same parameter in the corresponding orthostatic action in which the orthostatic load component, W, acts alone:

$$u'_n = \frac{Q_n}{Q_n - P} u_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (73)$$

or,

$$u'_n = u_n + \frac{P}{Q_n - P} u_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (74)$$

If u_n in Formula (73) or Formula (74) is considered as a constant, then the curve, (u'_n, P), in rectangular co-ordinates becomes a hyperbola with a horizontal asymptote, $P = Q_n$. The upper branch of this hyperbola represents an unstable equilibrium. If u_n is positive, then the lower branch of the hyperbola gives positive values, and the upper branch negative values, of u'_n. The reversal of the sign indicates that the equilibrium represented by points of the upper branch can be produced only when compensating deflections have been introduced first; that is, there must be deflections opposite those which are caused by W alone and which are represented by u_n.

By substituting Formula (74) in the series, Formula (60), which expresses the "effects" in the structure, and by using the prime to designate the action as heterostatic, the following formula will be found, which expresses the effect, F', in the heterostatic action in terms of the corresponding effect, F, in the orthostatic action produced by the orthostatic load component, W, acting alone:

$$F' = F + \Sigma \frac{P}{Q_n - P} F_n u_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (75)$$

This formula furnishes a solution of the problem of heterostatic action as existing in a large group of cases. An important example of application is the general Formula (12) in Article 7, which was used extensively in Part II, and which expresses bending moments in combined column action and bending, namely,*

$$M' = M + \Sigma \frac{P}{Q_n - P} M_n$$

As referred to previously, a part of the resilience of a straight column is due to the axial shortening, but as has been mentioned in connection with an application of Formula (70), and as will be shown in the following articles, this condition does not interfere with the applicability of the general Formula (75) to the case represented by Formula (12). One derives Formula (12) from Formula (75), by taking the bending moments, M and M', as the "effects", F and F', in Formula (75), and by substituting the bending moment, M_n, which is produced in the buckling under the axial load, Q_n, for the effect, $F_n u_n$, which is produced in the general astatic action under the load, Q_n.

* Goupil in *Annales des Ponts et Chaussées*, 1912, V, p. 401, indicates a formula and a rule of computation which might be derived as another special case of Formula (75). The case is that of a circular ring with constant cross-section, or of a cylinder with constant thickness. The load consists of pressures from the inside or outside, distributed in any manner. The "effects" are the deflections, but it is also shown how the bending moments may be found. The formula is similar to Formulas (75) and (12). It is evident that Formula (75), in the form of Formula (12), applies not only to straight, but also to curved members. Goupil quotes this derivation as having been indicated in lectures by Marbec and Le Besnerais at l'Ecole du Génie Maritime. This derivation is based on the differential equation of flexure applying in the particular case, and use is made of the Fourier series.

18.—Independent Parameters.—We shall now leave the simplified case treated in Article 17 and consider a case of a general nature. Assume that P denotes any load acting on the structure, and that no other loads are applied. Then the energy equation takes the form of Formula (39), or,

$$L = U - P\delta = \text{min.}$$

Assume that the resilience, U, can be expressed:

$$U = \bar{U}\ (u_1 \ldots\ldots u_n \ldots\ldots) + T\ (t_1 \ldots\ldots t_i \ldots\ldots) \ldots\ldots\ldots (76)$$

where $\bar{U}$ is a quadric in $u_1 \ldots\ldots u_n \ldots\ldots$, and T a quadric in $t_1 \ldots\ldots t_i \ldots\ldots$ As $\bar{U}$ cannot be negative, neither U nor T can be negative. It is assumed that $T = 0$ only when $t_1 = \ldots\ldots = t_i = \ldots\ldots = 0$. The displacement, δ, in the direction, P, is in the general case a quadratic function of the parameters. Assume that δ depends on the u's only, or,

$$\delta = \delta\ (u_1 \ldots\ldots u_n \ldots\ldots)$$

Under these circumstances, the general energy equation, Formula (39), is equivalent to the two independent conditions combined, namely,

$$T = \text{min.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (77)$$

and,

$$L' = \bar{U} - P\delta = \text{min.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (78)$$

Under the assumptions made, the only solution of Formula (77) is $t_1 = \ldots\ldots = t_i = \ldots\ldots = 0$. Formula (78) is an energy equation of the usual type shown in Formula (39), but contains only the parameters, $u_1 \ldots\ldots u_n$. Under these conditions, the parameters, $t_1 \ldots\ldots t_i$, are "parameters independent of the load, P". If P is an astatic load, or the astatic component of a load of general type, the parameters, $t_1 \ldots\ldots t_i$, will be referred to simply as "independent parameters".

When P is an astatic load, the occurrence of independent parameters may be represented as a limiting case in which one or more of the coefficients, δ_m, δ_n, $\delta_{mn} \ldots$, in the expressions for δ, such as Formula (50) or Formula (63), become equal to zero without the vanishing of the corresponding coefficients in U. When u_m is an astatic parameter, the corresponding astatic load is expressed, in general, as $Q_m = \dfrac{U_m}{\delta_m}$ (Formula (55)). Hence, $\delta_m = 0$ represents the limiting case, $Q_m = \infty$, the case in which the astatic load, Q_m, has become infinite. In the same manner, $\delta_m = \delta_n = \delta_{mn} = 0$ represents the limiting case, $Q_m = Q_n = \infty$.

As an example, consider an hinged-end column which is originally vertical and is supported at the ends by two horizontal springs, as indicated in Fig. 1(F). A horizontal parallel motion of the column would not change the vertical distance between the ends, therefore, a parameter expressing such a variation of position is an independent parameter. When the two springs are equally stiff, this parameter can be measured in any state of deformations, as the sum of the horizontal displacements of the two ends, considered positive in the same direction. Another parameter which is measured by the difference of the horizontal deflections of the ends, is a regular astatic parameter. The

functions which represent the internal actions corresponding to those two parameters are easily shown to be orthogonal, as they should be, because one corresponds to an infinite, and the other to a finite, critical astatic load.

As another example, take a structure consisting of a column plus some independent structure. The parameters defining the variations in this independent structure are independent parameters with reference to the axial load on the column.

19.—The Orthostatic Parameter in Pure Astatic Action, and the Vanishing of Certain Constants.—Consider the vertical hinged-end column, of a length, l, with a uniform stiffness factor, EI, an area of cross-section, A, and with the axial loads transferred at the ends only. It was pointed out in Article 17 that the coefficients, u_n, in the expression for the deflections, $y = \Sigma u_n \sin \frac{n \pi x}{l}$, combined with one parameter measuring the axial shortening, would form a complete set of parameters in terms of which any possible type of deformation could be expressed. At the beginning of Article 17, it was stated that if the column is very slender, the axial shortening would have only a negligible influence on the total energy variations, and in that case the column would belong directly in the simplified class treated in Article 17. It was also stated—without proof—that certain applications of the formulas derived could be made even if an appreciable amount of energy is consumed by the axial shortening. The questions involved here will now be investigated.

Let v denote the parameter which measures the axial shortening; that is, v is a quantity proportional to the uniformly distributed axial shortening due to the direct compressive stress. When the terms depending on v have been added, the resilience, U, and the displacement, δ, in the direction of P, are expressed as follows:

$$U = \frac{1}{2} \Sigma U_n u_n^2 + \frac{1}{2} V v^2$$

$$\delta = \frac{1}{2} \Sigma \delta_n u_n^2 + \gamma v$$

in which V and γ are constants. The energy equation, $L = U - P\delta = \text{min.}$, gives, in this case, $\frac{\partial L}{\partial v} = V v - P\gamma = 0$, or, $v = \frac{P\gamma}{V}$, that is, v is proportional to the load.

Also, in other cases of astatic action, there is generally one variation of shape which is directly proportional to the load and which can be expressed by a parameter such as v. In the special solution, in which all the astatic parameters are zero, the deformations are, as a whole, proportional to v and thereby to the astatic load, P. Such a parameter, which measures that particular variation in the astatic action which is proportional to the load, will be called an orthostatic parameter.

A general case will be considered in which the variations of the structure can be expressed in terms of the following three kinds of parameters: Astatic parameters, $u_1 \ldots\ldots u_n \ldots\ldots$; the orthostatic parameter, v; and the independent parameters, $t_1 \ldots\ldots t_i \ldots\ldots$ When these parameters are sufficient to define all the possible variations of the structure, the astatic action produced by the astatic

load, P, corresponding to the parameters, $u_1 \ldots\ldots u_n$, will be called a pure astatic action. This action is characterized by the condition that $P = Q_n$ gives as a possible solution, $u_n =$ any value, v being proportional to Q_n and all the other parameters being zero.

The energy equation, Formula (39), will be used again,

$$L = U - P\delta = \min.$$

In this case, we write:

$$U = U(u_1 \ldots\ldots u_n \ldots) + \frac{1}{2} V v^2 + v \Sigma V_n u_n + T(t_1 \ldots\ldots t_i \ldots\ldots) \ldots\ldots (79)$$

and,

$$\delta = \bar{\delta}(u_1 \ldots\ldots u_n \ldots) + \Sigma \varepsilon_n u_n + \gamma v + v \Sigma \gamma_n u_n \ldots\ldots\ldots (80)$$

in which $\bar{U}$, T, and $\bar{\delta}$ are quadrics, while V, V_n, ε_n, γ, and γ_n are constants. Differentiation gives:

$$\frac{\partial L}{\partial u_n} = \left(\frac{\partial \bar{U}}{\partial u_n} - P \frac{\partial \bar{\delta}}{\partial u_n}\right) - P \varepsilon_n + (V_n - P \gamma_n) v = 0 \ldots\ldots (81)$$

and,

$$\frac{\partial L}{\partial v} = (V v - P \gamma) + \Sigma (V_n - P \gamma_n) u_n = 0 \ldots\ldots\ldots (82)$$

Since $u_1 = \ldots\ldots u_n = \ldots\ldots = 0$ is a possible solution at any value of P, Formula (82) gives $V v - P \gamma = 0$, or,

$$v = \frac{P \gamma}{V} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (83)$$

$P = Q_n$ gives as a possible solution $u_n =$ any value, other u's being zero, therefore, Formula (82) gives $V_n = Q_n \gamma_n$. All u's being equal to zero makes $\frac{\partial \bar{U}}{\partial u_n} - P \frac{\partial \bar{\delta}}{\partial u_n} = 0$, that is, in this solution the first term in Formula (81) vanishes. When, especially, $P = Q_n$, the last term in Formula (81) vanishes, therefore, $Q_n \varepsilon_n = 0$, that is,

$$\varepsilon_1 = \ldots\ldots\ldots = \varepsilon_n = \ldots\ldots = 0 \ldots\ldots\ldots\ldots (84)$$

Finally, the combination, $u_1 = \ldots\ldots u_n = \ldots\ldots = 0$, v being finite, $P \neq Q_n$, gives, on account of Formula (81):

$$V_n = \gamma_n = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (85)$$

Formulas (84) and (85) express the result that, in the case of pure astatic action, the linear terms in the astatic parameters vanish in the expression for δ, and the mixed terms containing products of the orthostatic and the astatic parameters vanish in the expression for U and δ.

20.—The Parameters and the Energy Equation in the General Case.— A case will now be treated which is more general than the simplified one analyzed in Article 17. As in Article 17, the load will be assumed to consist of two component parts: $W =$ orthostatic component of the load, and $P =$ astatic component of the load.

As before, it is assumed that the series, $Q_1, Q_2 \ldots\ldots Q_n \ldots\ldots$, is the series of different critical values of the astatic load, P. It is also assumed that some

finite number of astatic parameters corresponds to each critical load, so that the complete set of astatic parameters can be written:

$u_{1a}, u_{1b} \ldots\ldots\ldots\ldots\ldots u_{1k}, \ldots\ldots\ldots$ corresponding to Q_1;
$u_{2a}, u_{2b} \ldots\ldots\ldots\ldots\ldots u_{2k}, \ldots\ldots\ldots$ corresponding to Q_2;
$\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$
$u_{na}, u_{nb} \ldots\ldots\ldots\ldots\ldots u_{nk}, \ldots\ldots\ldots$ corresponding to Q_n;
$\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$

In addition, there are the following special parameters: v = orthostatic parameter; and $t_1, t_2 \ldots\ldots t_i \ldots\ldots$ = independent parameters.

It is assumed that these parameters are sufficient to determine the possible variations of shape of the structure. It follows then from the definition given in Article 19 that the buckling under the influence of P is a pure astatic action. A prime will be used again to denote the particular value of a parameter in the heterostatic action; so that we have, for instance, u'_{nk} = the particular value of the parameter, u_{nk}, in the heterostatic action. The notation, δ, w, R, R', ds, r, F, and F', is the same as that given in Article 16.

The energy equation takes again the form of Formula (61), or,

$$L = U - P\delta - Ww = \text{min.}$$

When writing expressions for U and δ use will be made of the theorem of the vanishing of the mixed terms which contain the products of two astatic parameters that belong to different critical loads (Article 15). The coefficients of certain other terms also vanish, as was indicated in Article 19. The result is that the resilience, U, and the displacements, δ and w, can be expressed as follows:

$$U = \Sigma U^{(n)} + \frac{1}{2} Vv^2 + T(t_1, t_2 \ldots) \ldots\ldots\ldots\ldots\ldots\ldots (86)$$

$$\delta = \Sigma \delta^{(n)} + \gamma v \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (87)$$

$$w = \sum_n \sum_i w_{ni} u_{ni} + \bar{w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (88)$$

in which $U^{(n)}$ and $\delta^{(n)}$ are quadrics in the parameters, $u_{na} \ldots\ldots u_{ni} \ldots\ldots$, which belong to the critical load, Q_n, and in which,

V, γ, w_{ni} = constants;
T = quadric in the parameters, $t_1, t_2 \ldots\ldots$; and
w = linear function of $t_1, t_2 \ldots\ldots$, and v.

The quadric, $\delta^{(n)}$, may be written:

$$\delta^{(n)} = \frac{1}{2} \sum_i \delta_{ni} u_{ni}^2 + \Sigma \delta_{ni, nk} u_{ni} u_{nk} \ldots\ldots\ldots\ldots\ldots (89)$$

The relation, Formula (58), between the coefficients in U and δ applies to the coefficients in Formula (89), therefore, the quadric, $U^{(n)}$, can be written in the form:

$$U^{(n)} = Q_n \delta_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (90)$$

21.—Solution of the Energy Equation in the General Case.—

A.—Astatic Action.

Assume $P = Q_n$, $W = 0$, then the energy equation, Formula (61), is satisfied when $u_{na}, u_{nb} \ldots\ldots\ldots u_{ni} \ldots\ldots$ assume any values, and $v = P\frac{\gamma}{V}$ (Formula 83), all other parameters being equal to zero.

B.—Orthostatic Action.

$P = 0$ puts the energy equation in the form $L = U - Ww = \min$. Differentiation then gives a set of equations of the type:

$$\frac{\partial L}{\partial u_{ni}} = Q_n \frac{\partial \delta^{(n)}}{\partial u_{ni}} - Ww_{ni} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (91)$$

or, if Formula (89) is used:

$$Q_n (\delta_{na,ni}\ u_{na} + \delta_{nb,ni}\ u_{nb} + \ldots + \delta_{ni}\ u_{ni} + \ldots) = Ww_{ni} \ldots\ldots (92)$$

By making $i = 1, 2, 3 \ldots\ldots$, a set of equations similar to Formula (92) is obtained. One such set of linear equations is derived for each critical load, and thereby sufficient relations are found for the determination of all the astatic parameters. In the special case in which there is only one astatic parameter, u_n, corresponding to Q_n, the quadric, $\delta^{(n)}$, is reduced to the form:

$$\delta^{(n)} = \frac{1}{2}\ \delta_n u^2{}_n$$

whereby Formula (92) is reduced to the following special form, which is equivalent to Formula (69):

$$Q_n\ \delta_n u_n = Ww_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (93)$$

The orthostatic parameter, v, and the independent parameters, $t_1, t_2 \ldots\ldots$, are found in a similar manner from equations of the form:

$$\frac{\partial L}{\partial v} = 0, \frac{\partial L}{\partial t_1} = 0, \frac{\partial L}{\partial t_2} = 0, \text{etc.}$$

C.—Heterostatic Action.

In this case, the astatic parameters are denoted with a prime, as, for example, u'_{ni}. By substituting Formulas (86), (87), (88), and (90), in the energy equation, $L = U - P\delta - Ww = \min.$, and differentiating with respect to the astatic parameters, equations of the following type are found:

$$\frac{\partial L}{\partial u'_{ni}} = (Q_n - P) \frac{\partial \delta^{(n)}}{\partial u'_{ni}} - Ww_{ni} = 0 \ldots\ldots\ldots\ldots (94)$$

These equations are linear. By the substitution of Formula (89) for $\delta^{(n)}$, Formula (94) is transformed into:

$$(Q_n - P)(\delta_{na,ni}\ u'_{na} + \delta_{nb,ni}\ u'_{nb} + \ldots + \delta_{ni}\ u'_{ni} + \ldots) = Ww_{ni} \ldots (95)$$

A set of equations like Formula (95) is obtained by making $i = 1, 2, 3 \ldots\ldots$ One such set of equations belongs to each astatic load. By comparing Formula (95) with Formula (92), the following general relation is found between the values, u_{ni}, of the parameters in the orthostatic action produced by W alone,

and the values, u'_{ni}, of the same parameters in the heterostatic action produced by P and W together:

$$u'_{ni} = \frac{Q_n}{Q_n - P} u_{ni} \quad \ldots\ldots (96)$$

or,

$$u'_{ni} = u_{ni} + \frac{P}{Q_n - P} u_{ni} \quad \ldots\ldots (97)$$

These formulas are actually the same as Formulas (73) and (74), which were derived for the simplified case. That is, Formulas (73) and (74) can be applied generally.

The orthostatic parameter in the astatic action was found to be $v = \frac{P\gamma}{V}$ (Formula (83)). Assume that the orthostatic action gives the value, v_0. Since v enters only in linear terms in δ and w (Formulas (87) and (88)), the value, v', of the orthostatic parameter in the heterostatic action can be found by adding the values in the corresponding astatic and orthostatic actions, that is,

$$v' = \frac{P\gamma}{V} + v_0 \quad \ldots\ldots (98)$$

The derivatives, $\frac{\partial L}{\partial t_1}$, $\frac{\partial L}{\partial t_2}$, etc., are the same as those in the orthostatic action, hence the independent parameters, $t_1, t_2, \ldots\ldots$, are the same in the heterostatic action as in the corresponding orthostatic action.

Now, consider some effect, F. The particular value of F in the heterostatic action is denoted by F'. Assume that F can be expressed as,

$$F = \Sigma F^{(n)} + F_0 v + \bar{F}(t_1, t_2 \ldots) \quad \ldots\ldots (99)$$

where F_0 is a constant and where $F^{(n)}$ is a linear homogeneous function of the astatic parameters belonging to Q_n. That is, $F^{(n)}$ can be written:

$$F^{(n)} = F_{na} u_{na} + F_{nb} u_{nb} + \ldots + F_{ni} u_{ni} + \ldots\ldots (100)$$

We now introduce the particular values, $\ldots u'_{ni}$ and v', given by Formulas (97) and (98), in Formulas (99) and (100), and we find thereby the value, F', of the effect, F, in the heterostatic action. This value, F', is then expressed as follows in terms of the corresponding values, F, u_{ni} and $F^{(n)}$, which occur in the orthostatic action:

$$F' = F + \sum_n \frac{P}{Q_n - P} \sum_i F_{ni} u_{ni} + P \frac{F_0 \gamma}{V} \quad \ldots\ldots (101)$$

or,

$$F' = F + \sum_n \frac{P}{Q_n - P} F^{(n)} + P \frac{F_0 \gamma}{V} \quad \ldots\ldots (102)$$

Formulas (101) and (102) express the effect in the heterostatic action in a case of a most general character, namely, the general case in which the heterostatic load can be resolved into one orthostatic component, W, and one component, P, giving pure astatic action. Like F and u_n in Formula (75) (which is the general formula derived in the simplified case), F, u_n, and $F^{(n)}$ in Formulas (101) and (102) are values belonging to the "corresponding ortho-

static action" produced when $P = 0$. Formula (75) may be derived as the special case of Formula (102) in which $F^{(n)} = F_n\, u_n$ and $F_0\, \gamma = 0$. The general Formula (12) used in Part II is seen readily to be the special case of Formula (102), in which the "effects", F, F', and $F^{(n)}$, are the bending moments, M, M', and M_n, respectively. F_0 is zero in that special case. These considerations also prove finally that the applications made in Article 17 to straight columns are valid whether or not the axial shortening is appreciable.

22.—Determination of the Parameters in the General Case, When the Internal Actions Are Known.—The intensity of internal action, the generalized stress, R, may be considered as an "effect", and may be expressed in the same manner as F in Formula (99). We write then:

$$R = \Sigma R^{(n)} + R_0\, v + R\,(t_1, t_2 \ldots) \quad \ldots\ldots\ldots\ldots\ldots (103)$$

in which,

$$R^{(n)} = R_{na}\, u_{na} + R_{nb}\, u_{nb} + \ldots + R_{ni}\, u_{ni} + \ldots\ldots\ldots\ldots\ldots (104)$$

The energy of the element, ds, is $\frac{1}{2}\, rR^2 ds$, so that the total resilience is $U = \frac{1}{2} \int rR^2 ds$ (Formula (41)). U is a quadric in the parameters, but has no terms containing products of the following types: $u_{ni} u_{mk}$, where $n \neq m$; $u_{ni}\, v$; $u_{ni}\, t_k$; $v t_k$. This property of u is expressed in Formula (86), which omits all such terms. It follows that the property of orthogonality, proved in Article 15 with reference to two astatic parameters, is extended so as to include the following relations:

$$\int rR_{ni}\, R_{mk}\, ds = 0 \quad (m \neq n, \text{ compare Formula (57)}) \ldots\ldots\ldots (105)$$

$$\int rR_{ni}\, R_0\, ds = 0\,; \quad \int rR_{ni}\, \bar{R} ds = 0\,; \quad \int rR_0\, \bar{R} ds = 0 \ldots\ldots (106)$$

Multiplying Formula (103) by $rR_{ni}\, ds$ and integrating so as to include all elements of energy, thereby making use of Formulas (105) and (106), we find,

$$\int r\, R\, R_{ni}\, ds = \int r\, R^{(n)}\, R_{ni}\, ds \quad \ldots\ldots\ldots\ldots\ldots (107)$$

or,

$$\int r\, R\, R_{ni}\, ds = u_{na} \int r\, R_{na}\, R_{ni}\, ds + u_{nb} \int r\, R_{nb}\, R_{ni}\, ds + \ldots\ldots$$
$$+ u_{ni} \int r\, R_{ni}^{\,2}\, ds + \ldots\ldots\ldots\ldots\ldots (108)$$

The latter equation (Formula (108)) is one of a set which may be written:

$$\left.\begin{aligned} u_{na} \int r\, R_{na}^{\,2}\, ds + u_{nb} \int r\, R_{nb}\, R_{na}\, ds + \ldots\ldots \\ + u_{ni} \int r\, R_{ni}\, R_{na}\, ds + \ldots\ldots = \int rR\, R_{na}\, ds \\ u_{na} \int r\, R_{na}\, R_{nb}\, ds + u_{nb} \int r\, R_{nb}^{\,2}\, ds + \ldots\ldots \\ + u_{ni} \int r\, R_{ni}\, R_{nb}\, ds + \ldots\ldots = \int rR\, R_{nb}\, ds \\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \end{aligned}\right\} \quad (109)$$

This set contains one equation for each of the parameters, $u_{na}, u_{nb}, \ldots u_{ni} \ldots$, which belong to the critical load, Q_n. One set of such equations can be derived for each different critical load, and thereby a number of equations are found, which are sufficient for the determination of all the astatic parameters. A very important special case is that in which u_{na} is the only parameter belonging to the critical load, Q_n. In that case the set, Formula (109), is reduced to one equation, which may be written:

$$u_{na} = \int \frac{r\, R\, R_{na}\, ds}{r\, R_{na}^{\,2}\, ds} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (110)$$

The index, n, may be used in this special case instead of na. Then, Formula (110) becomes identical with Formula (59), which was derived for the simplified case in Article 17, in a very similar manner.

Formula (110) applies under special circumstances, even when there is more than one astatic parameter belonging to Q_n. It applies whenever the condition, $\int r\, R_{nb}\, R_{na}\, ds = \ldots = \int r\, R_{ni}\, R_{na}\, ds = \ldots = 0$, is satisfied. In such a case, the parameter, u_{na}, will be said to have been orthogonalized with respect to the other parameters in the group, $u_{nb} \ldots u_{ni}$. It is always possible, by a proper choice of the parameters in the group belonging to one critical load, to orthogonalize all the parameters in this group with respect to each other. This process is often facilitated by properties of symmetry of the structure. Take, for instance, the vertical hinged-end column with spherical bearings and with circular cross-section: Two astatic parameters will correspond to each critical load. Each such pair of parameters is orthogonalized by choosing the parameters so that they measure the amounts of buckling in two vertical planes perpendicular to one another.

The orthostatic parameter, v, is found in a similar way. Multiplying Formula (103) by $r\, R_0\, ds$, integrating, thereby using the properties of orthogonality expressed in Formula (106), and solving for v, we find an expression, similar to Formula (110), namely,

$$v = \int \frac{r\, R\, R_0\, d\, s}{r\, R_0^{\,2}\, d\, s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (111)$$

The independent parameters, $t_1, t_2 \ldots\ldots$, can be determined in the same manner as any group of astatic parameters. One finds a set of equations of the same form as that shown in Formula (109).

23.—Mixed Astatic Action.—We will consider again the general formula for heterostatic action, namely,

$$F' = F + \sum_n \frac{P}{Q_n - P}\, F^{(n)} + P\, \frac{F_0\, \gamma}{V} \quad \ldots\ldots\ldots\ldots\ldots \quad (102)$$

where $F^{(n)} = \Sigma\, F_{ni}\, u_{ni}$. In the special case, $P = Q_n$, the effect, F', would generally become infinite, except when $F^{(n)} = 0$, in which particular case, Formula (102) indicates F' as indefinite. This consideration suggests that P combined with special types of the orthostatic load, W, may cause astatic actions which differ from the pure astatic action produced by P acting alone, but which have, nevertheless, some of the characteristic features of pure buckling. The most important of these features is that of the indefinite dis-

placements at constant values of the load. The possibility of such "mixed astatic actions", produced under the combined influence of a pure astatic load and an orthostatic load, will now be examined.

Consider Formula (95): It is one of a set belonging to Q_n. The set is completed by exchanging the indices, $na, nb \ldots\ldots$, respectively, for ni. It is assumed that,

$$w_{na} = w_{nb} = \ldots\ldots = w_{ni} = \ldots\ldots = 0.$$

Then the set of Formulas (95) is solved as follows:

$$\text{When } P \neq Q_n,\ u'_{na} = u'_{nb} = \ldots\ldots\ u'_{ni} = \ldots\ldots = 0;$$
$$\text{When } P = Q_n,\ u'_{na}, u'_{nb} \ldots\ldots\ u'_{ni} = \ldots\ldots = \text{any values.}$$

This solution satisfies Formula (94), and when the other parameters have been found by the methods indicated in Article 21, by using, for instance, Formulas (97) and (98), the combined solution will satisfy the general energy equation, Formula (61). The conclusion is that $P = Q_n$ produces astatic action when combined with W, provided the coefficients, $w_{na} \ldots\ldots w_{ni} \ldots\ldots$, vanish. On account of Formula (92), this condition can also be expressed as follows: Astatic action is possible when the values, $u_{na}, u_{nb} \ldots\ldots u_{ni} \ldots\ldots$, as determined in the orthostatic action produced by W alone, are zero.

A whole series of special critical loads, $Q_n, Q_n', Q_n'' \ldots\ldots$, forming a part of the complete series of critical values of P, may give astatic action, although the astatic load is combined with the orthostatic load, W. Now, assume that W varies proportionally to P. In that case, the combination, P, W, can be considered as one load, the intensity of which is measured by P. As it produces astatic action when the intensity reaches the critical values, $Q_n, Q_n', Q_n'' \ldots\ldots$, the heterostatic load, P, W, may be considered as an astatic load. Let u_{mk} be some astatic parameter which W, acting alone, causes to be different from zero, and let the value, u_{mk}, be the particular value produced by W alone; then, Formula (97) would give the corresponding value of the parameter produced by the combination, P, W, namely,

$$u'_{mk} = u_{mk} + \frac{P}{Q_m - P} u_{mk}$$

u_{mk} is proportional to W, which is assumed to be proportional to P. It follows that u'_{mk} is neither constantly zero, nor proportional to P, and therein lies a principal difference between the pure astatic action produced by P, and astatic action produced by the load, P, W. It is the latter type of astatic action which is designated as "mixed astatic action". This discussion may be summarized as follows: The load, P, W, in which W is proportional to P, may produce astatic action at a series of critical values of P. When one or more of the astatic parameters becomes different from zero, under the influence of W, then stresses and other effects in the structure are neither all zero, nor proportional to the load, even when the load is different from any critical value. In that case, the action is a mixed astatic action, to be distinguished from the pure astatic action in which, as long as the load is different from its critical values, the effects are either zero or proportional to the load.

In the case analyzed, $u_{na}, u_{nb} \ldots\ldots u_{ni} \ldots\ldots$ are astatic parameters, not only with reference to the load, P, but also with reference to the astatic load, P, W.

Yet, u_{mk} can be classified neither as an astatic, an orthostatic, nor as an independent parameter. The only case in which the effects in a structure can be described completely in terms of astatic parameters, one orthostatic parameter, and independent parameters, with reference to a certain load, is that in which this load produces pure astatic action. In most cases, it is possible to conclude from the nature of the structure and from the nature of the load whether a certain astatic action is a pure or a mixed action. If it is a mixed astatic action, then, in order to make the general Formulas (101) and (102) applicable, the load should be resolved into two components, one, P, giving pure astatic action, the other, W, giving orthostatic action.

As an example, consider a hinged-end column with the load, P, transferred at the ends with equal but opposite eccentricities, $\pm e$. The case was treated in Article 9. Astatic equilibria occur when $P = \frac{n^2 \pi^2 E I}{l^2}$, where $E I =$ the stiffness factor, $l =$ the length, and $n = 1, 3, 5 \ldots\ldots$ Between these critical loads, the stresses, deflections, and other actions do not increase proportionally to the load, except at special points. The case may be classified as heterostatic action or as mixed astatic action. The load may be resolved into an astatic component giving pure astatic action, namely, a central end load, P, and an orthostatic component, namely, the end moments, Pe, both clockwise, or both anti-clockwise. If the same column is loaded by a central axial end load and, at the same time, by a symmetrical transverse load, then astatic equilibria occur at $P = \frac{n^2 \pi^2 E I}{l^2}$, with $n = 2, 4, 6 \ldots\ldots$ Also, this case is evidently one of mixed astatic action.

24.—Initial Eccentricities in a Simplified Case.—A structure is considered, which depends on the parameters, $u_1 \ldots\ldots u_n$, which are all astatic parameters with respect to the load, P. When all the critical loads, $Q_1 \ldots\ldots Q_n$, are different, then U and δ of the energy equation, $L = U - P\delta = \text{min.}$, can be expressed as in Formulas (66) and (63) in Article 17,

$$U = \frac{1}{2} \Sigma Q_n \delta_n u_n^2 ; \qquad \delta = \frac{1}{2} \Sigma \delta_n u_n^2$$

Now, assume a change of shape defined by $u_1 = e_1, \ldots\ldots u_n = e_n \ldots$, and assume that these deformations remain as a "permanent set" after the release of the load. It is also assumed that zero stresses correspond to zero load after the change has taken place, and that all other elastic properties, stiffness constants, etc., remain as before the change. In the changed structure, the original load, P, will no longer produce pure astatic action. The new case can be described as differing from the original case by eccentricities which are measured by $e_1 \ldots\ldots e_n \ldots\ldots$ By definition, $e_1 \ldots\ldots e_n \ldots\ldots$ will be called "the eccentricities", or, more explicitly, "the eccentricities of the parameters, $u_1 \ldots\ldots u_n \ldots\ldots$, with respect to the astatic load, P". The two structures, the original and the changed, will be called the "concentric" and the "eccentric" structures, respectively.

The quantities, $u_1 \ldots\ldots u_n \ldots\ldots$, can be used as parameters of both the eccentric and the concentric structure. The method of measuring the param-

eters starting from the initial shape of the concentric structure, will first be used; that is, in the eccentric structure, $u_1 = e_1, \ldots\ldots u_n = e_n \ldots\ldots$, represents the state of zero stresses, while $u_1 = \ldots = u_n = \ldots = 0$ indicates the shape which coincides with the initial shape of the concentric structure. This is one of the exceptional cases, in which the general rule laid down in Article 11, that zero values of all the parameters represent zero stresses, is not followed. As the stiffness constants are the same in both structures, the resilience quadric of the eccentric structure becomes:

$$U = \frac{1}{2} \Sigma\, Q_n\, \delta_n\, (u_n - e_n)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (112)$$

When δ is measured, starting from the initial shape of the original structure, the expression, Formula (63), for δ remains unchanged, that is, we have also in the eccentric structure:

$$\delta = \frac{1}{2} \Sigma\, \delta_n\, u_n^2$$

Substituting Formulas (63) and (112) in the equation, $L = U - P\delta = \text{min.}$, one finds the energy equation of the eccentric structure:

$$L' = \frac{1}{2} \Sigma\, (Q_n - P)\, \delta_n\, u_n^2 - \Sigma\, Q_n\, \delta_n\, e_n\, u_n = \text{min} \ldots\ldots\ldots (113)$$

Differentiation with respect to u_n gives:

$$u_n = e_n + \frac{P}{Q_n - P}\, e_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots (114)$$

By comparing Formula (114) with the general Formula (74) derived in Article 17, it is seen that the value of the parameter, u_n, in the eccentric action can be found as the value of the same parameter in a certain heterostatic action which occurs in the original concentric structure, namely, the heterostatic action in which the astatic load, P, is combined with an orthostatic load which, if acting alone, would produce the initial shape of the eccentric structure. The character of this orthostatic load, W, and of its displacement, w, is defined by Formula (115):

$$W\, w = \Sigma\, Q_n\, \delta_n\, e_n\, u_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots (115)$$

Effects which are measured, starting from the initial shape of the original structure, such as deflections from this initial shape, can be found by substituting the values of $u_1 \ldots\ldots u_n \ldots\ldots$, found by Formula (114), in the formulas which determine these effects in the original structure. Other effects, such as stresses or deflections from the initial shape of the eccentric structure, effects which are zero when $u_1 = e_1, \ldots\ldots u_n = e_n \ldots\ldots$, are found by substituting $u_1 - e_1, \ldots\ldots u_n - e_n \ldots\ldots$ for $u_1 \ldots\ldots u_n \ldots\ldots$, in the formulas for these effects in the original structure.

This result suggests that, in certain cases, it might be advantageous to measure the parameters starting not from the initial shape of the original structure, but from the initial shape of the eccentric structure. Let the parameters measured in this manner be $\bar{u}_1 \ldots\ldots \bar{u}_n \ldots\ldots$ They are derived from the parameters, $u_1 \ldots\ldots u_n \ldots\ldots$, simply by subtracting the eccentricities,

$e_1 \ldots\ldots e_n \ldots\ldots$; that is, $\ldots\ldots \bar{u}_n = u_n - e_n$, etc. Hence, the resilience is expressed as follows:

$$U = \frac{1}{2} \Sigma\, Q_n\, \delta_n\, \bar{u}_n^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (116)$$

By measuring δ as before, from the initial shape of the original structure, it is found that,

$$\delta = \frac{1}{2} \Sigma\, \delta_n\, (\bar{u}_n + e_n)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (117)$$

Substitution of Formulas (116) and (117) puts the energy equation, $L = U - P\delta = \text{min.}$, in the form:

$$L'' = \frac{1}{2} \Sigma \left(Q_n - P \right) \delta_n \bar{u}_n^2 - P \Sigma\, \delta_n e_n \bar{u}_n = \text{min.} \ldots\ldots \quad (118)$$

Differentiation with respect to $\bar{u}_n$ gives:

$$u_n = \frac{P}{Q_n - P}\, e_n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (119)$$

Since $u_n = \bar{u}_n + e_n$, Formula (119) is actually the same as Formula (114). Formula (118) leads to another interpretation of the eccentric action as a case of heterostatic action, and appears as the equation of a heterostatic action in which both the orthostatic and the astatic components of the load are measured in intensity by P. The orthostatic component is defined by its work, $Ww = P \Sigma\, \delta_n e_n \bar{u}_n$. The displacement in the direction of the astatic load in Formula (118) is expressed as $\delta' = \frac{1}{2} \Sigma\, \delta_n \bar{u}_n^2$, while Formula (117) expresses the displacement in the direction of the original load. The astatic component in Formula (118) may be obtained by adding certain loads proportional to P to the original load. Since the total given load, P, is the heterostatic load in Formula (118), however, the loads mentioned (which were to be added to the given load in order to produce the astatic load) will be equal and opposite to the loads which constitute the orthostatic component.

25.—Initial Eccentricities in the General Case.—Consider a structure of a more general type, using the notation given in Article 20. A load, P, is assumed to produce pure astatic action. A change or deformation is considered, in which the astatic parameters, $u_{na} \ldots\ldots u_{ni} \ldots\ldots$, belonging to the critical load, Q_n, take the values, $e_{na} \ldots\ldots e_{ni} \ldots\ldots$, and in which the other astatic parameters take values which are denoted correspondingly. These deformations are assumed to be retained as a permanent set after the release of the load, so that the changed shape is now that which corresponds to zero stresses and zero resilience. As in Article 24, the quantities, $e_{na} \ldots\ldots e_{ni} \ldots\ldots$, etc., are called, by definition, "the eccentricities of the parameters with respect to the load, P." The stiffness constants, such as the coefficients in the resilience quadric, are assumed again to remain the same as in the original structure. When the parameters and the deflections are measured from the original shape of the structure, that is, when the set of values, $u_{na} = e_{na} \ldots\ldots u_{ni} = e_{ni}$, etc., represents the initial shape of the changed eccentric structure, the expression

for the displacement, δ, in the direction of P will remain the same as in the original structure. According to Article 20, we have then Formula (87):

$$\delta = \Sigma\, \delta^{(n)} + \gamma\, v$$

where (Formula 89),

$$\delta^{(n)} = \frac{1}{2} \sum_{i} \delta_{ni}\, u_{ni}^{\,2} + \sum_{i,k} \delta_{ni,\,nk}\, u_{ni}\, u_{nk}$$

The expression for the resilience, U, is modified so as to make the set of values, $u_{na} = e_{na} \ldots\ldots u_{ni} = e_{ni} \ldots\ldots$, give $U = \text{min.}$, that is, instead of Formula (86), we have for the eccentric structure:

$$U = \Sigma\, \bar{U}^{(n)} + \frac{1}{2}\, V v^2 + T\,(t_1, t_2, \ldots\ldots) \ldots\ldots\ldots\ldots (120)$$

where,

$$\bar{U}^{(n)} = \frac{1}{2} \sum_{i} Q_n\, \delta_{ni}\, (u_{ni} - e_{ni})^2$$

$$+ \sum_{i,k} Q_n\, \delta_{ni,\,nk}\, (u_{ni} - e_{ni})\, (u_{nk} - e_{nk}) \ldots\ldots\ldots\ldots (121)$$

Substitution of Formulas (87), (89), (120), and (121) in the energy equation, $L = U - P\delta = \text{min.}$, and differentiation with respect to u_{ni} gives

$$\frac{\partial L}{\partial u_n} = \left(Q_n - P\right) \left(\delta_{ni}\, u_{ni} + \sum_{k} \delta_{ni,\,nk}\, u_{nk}\right)$$

$$- Q_n \left(\delta_{ni}\, e_{ni} + \sum_{k} \delta_{ni,\,nkn} e_{nk}\right) = 0 \ldots\ldots\ldots\ldots\ldots (122)$$

where $k \neq i$. By exchanging the indices, $na, nb \ldots\ldots$, for ni, a set of linear equations is obtained from which the parameters, $u_{na}, u_{nb}, u_{ni} \ldots\ldots$, belonging to Q_n can be determined. The solution of the set of Formula (122), is:

$$u_{na} = e_{na} + \frac{P}{Q_n - P}\, e_{na}, \ldots\ldots \quad u_{ni} = e_{ni} + \frac{P}{Q_n - P}\, e_{ni}, \ldots\ldots (123)$$

Formula (123) is virtually the same as Formula (114), derived for the simplified case. Formula (123) should be compared with Formula (97), the general formula for astatic parameters in heterostatic action. The comparison also shows that, in the case of the eccentric action in the structure of more general character, the values of the parameters can be determined as the values generated in a certain heterostatic action in the original structure; and as, in the simplified case, this heterostatic action is produced by the load, P, which is an astatic load in the original structure, combined with an orthostatic load which would produce the eccentricities, $e_{na} \ldots\ldots e_{ni}$, etc.

As in the simplified case, one may measure the parameters starting from the initial shape of the eccentric structure, that is, instead of u_{ni}, one may use, $\bar{u}_{ni} = u_{ni} - e_{ni}$. This method gives, analogous to Formula (119):

$$u_{ni} = \frac{P}{Q - P}\, e_{ni} \ldots\ldots\ldots\ldots\ldots\ldots (124)$$

With the quantities, $\bar{u}_{ni}$, introduced as parameters in the expressions for resilience and displacement, a form of the energy equation will be derived which is analogous to Formula (118). One finds, then, as in the simplified

case, that the eccentric action can be interpreted as a heterostatic action in the eccentric structure. In that action, both the orthostatic component and the astatic component of the load, are proportional to P, the astatic component differing from the load, P, on the original structure by a load equal and opposite to the orthostatic component.

If a permanent set causes $v = v_e$ to be the value of the orthostatic parameter at zero stresses, then v_e added to the value of v produced in the original structure, gives the value of v in the changed structure, as measured from the initial shape of the original structure. This conclusion is easily verified by an application of the energy equation. In the same manner, if a permanent set causes one of the independent parameters, say, t_1, to take the value, t'_1, at zero stresses, then, as long as P is the only load, $t_1 = t'_1$ will remain as the constant value of that parameter. In other words, permanent sets represented by changes of the orthostatic parameter and of the independent parameter can be treated independently, that is, the effects due to such permanent sets can be combined with the effects due to the eccentricities of the parameters, $u_{na} \ldots\ldots u_{ni} \ldots\ldots$, etc., by simple addition.

26.—Effects in Eccentric Action.—As a typical example, consider the hinged-end column, with a length, l, and a constant stiffness factor, EI. Deflections from the originally straight center line can be expressed as follows:

$$y = \Sigma u_n \sin \frac{n\pi x}{l},$$

where x is the distance from one end. The factors, u_n, will be used as the astatic parameters. The critical loads are expressed by the Euler formula,

$$Q_1 = Q = \frac{\pi^2 EI}{l^2}\ ; \quad Q_n = n^2 Q$$

The column, which had originally a straight center line, is now bent in such a way that permanent sets are left. The deflections, e, which remain when the load is released and the stresses are zero, may be expressed by a Fourier series of the form:

$$e = \Sigma e_n \sin \frac{n\pi x}{l}$$

According to the definitions made in Articles 24 and 25, $e_1 \ldots\ldots e_n \ldots\ldots$ are then the eccentricities of the astatic parameters, $u_1 \ldots\ldots u_n \ldots\ldots$, with respect to the end load, P. Denote, further: c = the distance from the center line to the extreme fibers, so that $\frac{I}{c}$ = the section modulus; A = the area of cross-section; and k = the radius of gyration, defined by $I = Ak^2$. The stresses, s, in the extreme fibers are among the effects which have particular interest, if y is measured from the original straight center line, which is also the line of the force, P; in that case, the stress is expressed as:

$$S = \frac{P}{A} + \frac{P\,yc}{I}\ ; \quad \text{or,} \quad S = \frac{P}{A}\left(1 + \frac{y\,c}{k^2}\right)$$

According to Formula (123) or Formula (114), we have:

$$u_n = e_n + \frac{P}{Q_n - P} e_n = e_n + \frac{P}{n^2 Q - P} e_n$$

By substituting these values in the expression for y, and then introducing the value of y in the expression for S, we find:

$$S = \frac{P}{A}\left(1 + \frac{c}{k^2}\left(e + \Sigma \frac{P}{n^2 Q - P} e_n \sin \frac{n\pi x}{l}\right)\right) \quad \ldots\ldots\ldots(125)$$

where $n = 1, 2, 3 \ldots\ldots$

Formula (125) gives the stress in the extreme fibers at any one point of the column. The reversed problem, that of finding the limiting value of P corresponding to a maximum allowable stress, S, may be treated by solving Formula (125), with respect to P. P is then found as a function of x and l. For each given value of l, the distance, x, has to be chosen so as to make P a minimum. In this manner, the limiting values of P will be found as a function of l. This function, say, $P = P(l)$, can be considered as the solution of Formula (125). In actual columns, it is impossible to secure an absolutely straight center line, an absolutely concentric loading, or a perfect evenness of the elastic qualities. The result is that eccentricities, such as those measured by e, or by $e_1, \ldots\ldots e_n$, cannot be avoided altogether in actual columns. With definite, properly chosen values of $e_1 \ldots\ldots e_n \ldots\ldots$, Formula (125), therefore, can be considered as a general column formula applying to actual columns, as long as their action depends essentially on the conditions before the elastic limit has been exceeded.* The column formulas commonly used in engineering design, such as that of Rankine, or the combination of the Euler formula with Johnson's parabolic formula, therefore, should appear as approximations to solutions of Formula (125). In the analysis of columns, one sometimes assumes a constant eccentricity which should then be considered as an "equivalent constant eccentricity", producing the same effect as the actual varying eccentricity. With e = constant, Formula (125) would lead to a solution, but the case is solved more easily by direct application of the differential equation of flexure. The result is the well known secant formula, which is given in many textbooks on Strength of Materials. With other end conditions, fixed ends, one free end, etc., formulas quite similar to Formula (125) can be derived. It may be said then that Formula (125) is a column formula of a general type, which applies to actual columns having small definite eccentricities.

If in the case solved by Formula (125), the greatest stress can be expected to be at or near the middle of the column, the solution can be simplified by introducing the definite value, $x = \frac{l}{2}$. This value gives $\sin \frac{n\pi x}{l}$ equal to: zero when n is even; $+1$ when $n = 1, 5, 9, \ldots\ldots$; and -1 when $n = 3, 7, \ldots\ldots$ Thereby, Formula (125) is reduced to:

$$S = \frac{P}{A}\left(1 + \frac{c}{k^2}\left(e_{center} + \frac{P}{Q - P} e_1 - \frac{P}{9Q - P} e_3 + \frac{P}{25Q - P} e_5 \ldots\right)\right) \quad (126)$$

where $e_{center} = e_1 - e_3 + e_5 \ldots\ldots$

* Compare Kármán's work in which the influence of stresses above the elastic limit is investigated, *Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, v. 81 (1910).

The general case of eccentric action in any structure may now be considered. It is assumed that the load, P, would produce pure astatic action if it was not for the eccentricities, e_{na}, e_{nb} ..... e_{ni} ....., etc., of the astatic parameters, u_{na}, u_{nb} ..... u_{ni} ....., etc. One may consider some "effect", F. F will be assumed to be an effect which is expressed by the same function of the parameters whether the original or the eccentric structure is considered, the parameters being measured in both cases, from the initial shape of the concentric structure. In the special case previously treated, the stress, S, when expressed as indicated in terms of the deflections, y, is an effect of this kind, and so is, in general, any effect which is proportional to the deflections from the original shape. The general effect, F, was expressed by Formula (99), which, after substitution of Formula (100), may be written in the form:

$$F = \sum_n \sum_i F_{ni}\, u_{ni} + F_0\, v + \bar{F}\,(t_1, t_2, \ldots\ldots) \qquad (127)$$

When P acts alone on the original concentric structure, and is different from any of the critical values — so that no buckling takes place — the effect is proportional to the orthostatic parameter, and may be written, in accordance with Formulas (127) and (83), as follows:

$$F_c = F_0\, v = P\,\frac{F_0\,\gamma}{V} \qquad (128)$$

Let t_1', t_2' ....., be the eccentricities caused in the independent parameters, t_1, t_2 ....., by the permanent set. By substituting these values, Formula (128), and the values of the parameters, u_{ni}, given by Formula (123), in Formula (127), the effect in the eccentric action, is found to be as follows:

$$F = F_c + \sum_n \sum_i F_{ni}\, e_{ni} + \sum_n \frac{P}{Q_n - P} \sum_i F_{ni}\, e_{ni} + \bar{F}\,(t_1', t_2', \ldots\ldots). \qquad (129)$$

Formula (129), as might be expected, is quite similar to Formula (101), which determines the effects, F', in heterostatic action. As a matter of fact, Formula (129) might have been derived as a special case of Formula (101).

If F is an effect which varies from point to point, then F and the coefficients, F_{ni}, are functions of the location. Assume that the limiting strength of the structure is defined by some limiting value of F, which must not be exceeded. The lowest value of P found by solving Formula (129), with this value of F substituted, determines the upper limit beyond which the load, P, cannot be increased. In other words, Formula (129), like Formula (125) in the special case, can be used as a general formula to define the carrying capacity in eccentric action. It is practically impossible to build any structure absolutely exactly as designed. It follows that small eccentricities such as the e's in Formulas (125) and (129), must always be expected to exist. Equations such as Formula (129), with certain assigned values of the eccentricities, therefore, may be used to determine the limiting values of loads which are apparently astatic loads, and would be so if the structure was built with absolute exactness. It follows from the similarity between Formulas (129) and (125) that in a great number of the cases covered by the general Formula (129), it will be possible to substitute approximate formulas similar to the

solutions of Formula (125); that is, formulas similar to that of Rankine, Johnson's parabolic formula, etc., will be approximate solutions of the general Formula (129) in various cases of buckling.

27.—Determination of Astatic Actions and Astatic Parameters by the Expressions for Resilience and Displacement.—Assume that a certain structure is defined with sufficient exactness by a finite number of parameters, $x_1, x_2 \ldots\ldots$, which are not assumed, in general, to be astatic parameters. Let the resilience be expressed by the quadric:

$$U = \frac{1}{2} A_1 x_1^2 + \frac{1}{2} A_2 x_2^2 + \ldots + A_{1,2} x_1 x_2 + \ldots\ldots\ldots \quad (130)$$

Assume that the displacement in the direction of the load, P, is also expressed by a quadric, namely,

$$\delta = \frac{1}{2} a_1 x_1^2 + \frac{1}{2} a_2 x_2^2 + \ldots + a_{1,2} x_1 x_2 + \ldots\ldots\ldots \quad (131)$$

Then, the energy equation, $L = U - P\delta = \text{min.}$, gives:

$$\left.\begin{aligned} \frac{\partial L}{\partial x_1} &= (A_1 - a_1 P) x_1 + (A_{1,2} - a_{1,2} P) x_2 \\ &\quad + (A_{1,3} - a_{1,3} P) x_3 + \ldots = 0 \\ \frac{\partial L}{\partial x_2} &= (A_{1,2} - a_{1,2} P) x_1 + (A_2 - a_2 P) x_2 \\ &\quad + (A_{2,3} - a_{2,3} P) x_3 + \ldots = 0 \\ &\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \end{aligned}\right\} \ldots\ldots\ldots (132)$$

The equations, Formula (132), are solved by $x_1 = x_2 = \ldots\ldots = 0$. Solutions different from zero are possible, however, when the determinant of the equations, Formula (132), vanishes,* that is, when:

$$\left.\begin{aligned} &A_1 - a_1 P \quad A_{1,2} - a_{1,2} P \quad A_{1,3} - a_{1,3} P \ldots \\ &A_{1,2} - a_{1,2} P \quad A_2 - a_2 P \quad A_{2,3} - A_{2,3} P \ldots \\ &\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \end{aligned}\right\} = 0 \ldots (133)$$

If there are m parameters, Formula (133) is an equation of mth degree in P, and has, in general, m solutions. Assume that these solutions are $P = Q_1$, $Q_2 \ldots\ldots Q_n \ldots\ldots$, and take, for instance, $P = Q_n$. Substituted in Formulas (132), $P = Q_n$ gives the set:

$$\left.\begin{aligned} &(A_1 - a_1 Q_n) x_1 + (A_{1,2} - a_{1,2} Q_n) x_2 + (A_{1,3} - a_{1,3} Q_n) x_3 + \ldots = 0 \\ &(A_{1,2} - a_{1,2} Q_n) x_1 + (A_2 - a_2 Q_n) x_2 + (A_{2,3} - a_{2,3} Q_n) x_3 + \ldots = 0 \\ &\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \end{aligned}\right\} \ldots (134)$$

With $x_1 = c_{n1} u_n$, where c_{n1} is a chosen constant, and u_n = any value, $m - 1$, of the equations, Formula (134), gives solutions which satisfy the remaining equation, hence, the solutions of the equations, Formula (134), may be expressed, as follows:

$$x_1 = c_{n1} u_n \; ; \quad x_2 = c_{n2} u_n \; ; \quad x_3 = c_{n3} u_n \; ; \ldots\ldots\ldots (135)$$

where u_n can take any value. u_n can be used as a parameter defining the variation expressed in Formula (135). Furthermore, u_n can be interpreted as

* The vanishing of a determinant is used by several writers as a criterion of buckling or of an equilibrium ceasing to be stable; for example, R. V. Southwell, *Philosophical Transactions*, Royal Soc., A, v. 213 (1914), p. 218; A. Ostenfeld, "Teknisk Statik", II, ed. 1913, p. 490; H. Zimmerman, *Sitzungsberichte* der. k. preuss. Akad. der Wissenschaften (1909), p. 193.

an astatic parameter. Its corresponding critical load is $P = Q_n$. By making $P = Q_1, Q_2 \ldots\ldots$, successively, and repeating the same operations, the set of astatic parameters, $u_1, u_2 \ldots\ldots$, may be determined completely.

One conclusion reached is that pure astatic action occurs when both the displacement in the direction of the load and the resilience are quadrics in the parameters. It is easy to verify the fact that this conclusion will continue to hold when additional terms, $\frac{1}{2} V v^2 + T(t_1, t_2 \ldots)$, in U, and γv in δ, are introduced. v and $t_1, t_2 \ldots$ will be recognized at once as orthostatic and independent parameters, respectively.

If δ had been a linear function of the parameters instead of a quadric, P would have been an orthostatic load. In a case of more general nature, the displacement in the direction of the load, P, is a quadratic function containing some linear terms. The total displacement may then be expressed as $\delta' = \delta + w$, where δ is a quadric of the form, Formula (131), while w contains the linear terms. In this case, the total load, P, may be interpreted as consisting of two components, each measured in intensity by the quantity, P. One component of P is an astatic load in the direction of the displacement, δ, the other component of P is an orthostatic load in the direction of the displacement, w. One concludes that if the resilience is a quadric in terms of a certain set of parameters, and if the displacement in the direction of a given load is expressed in terms of these parameters as a quadratic function containing linear terms, the action produced is a heterostatic action. This heterostatic action is recognized as one of the general type produced under the combined influence of an orthostatic and an astatic component of the load.

V.—Differential Equations Applying to Some Definite Cases.

28.—The Methods of Solution: Differential Equations and the Energy Method.—The problems treated herein are usually solved by one of two fundamental methods, one of which is based on the energy principle and the other on the direct statical principles of equilibrium. Both methods are represented at many places in the works mentioned in the Bibliography (Part VII). The general theory in Part IV of this investigation was based on the energy principle. The problem of equilibrium is reduced by the energy principle to a problem of energy minimum, or, to a variation problem. The analyses in Part IV show clearly that this method of attack is particularly effective in the derivation of general principles; but the history of technical statics has shown that the principle of least action can be applied with advantage also to structural problems of definite and specific character. The fundamental work of Castigliano* on statically indeterminate structures is an example. Castigliano's method of least work was applied primarily to problems the solution of which depends on a finite number of unknown or "indeterminate" quantities. In 1909, W. Ritz† indicated a method by which Castigliano's principles can be

* A. Castigliano, "Théorie de l'équilibre des systèmes élastiques et ses applications", Torino, 1879.

† W. Ritz, "Ueber eine neue Methode zur Lösung gewisser Variationsprobleme der mathematischen Physik", Crelles' *Journal für reine und angewandte Mathematik*, v. 135 (1909).

used even when the structure depends on an infinite number of "indeterminate" parameters. Results are then expressed in series giving successive approximations. Ritz's method has been applied since by several writers in the analysis of continuous structures, such as slabs, domes, water tanks, etc. S. Timoshenko* used the method extensively in the analysis of astatic action. The case of a column with varying section, for example, is analyzed conveniently by Ritz's method.† The conclusion is that the energy principle can be used with advantage in the analysis of the general questions and of some of the more complex specific questions.

Other cases are analyzed, however, as easily or more easily by the use of the direct statical principles of equilibrium. These principles are expressed in most cases of astatic or heterostatic action by certain differential equations which represent the conditions of equilibrium of each element of the structure. Thus, the solution of the problem is made to depend on the solution of certain differential equations. This method is advantageous in the analysis of the simple structures, such as single columns, curved members, and slabs. In the following Articles, the derivation of the differential equations of heterostatic action in such structures will be indicated. By omitting either the astatic or the orthostatic component of the load, one may analyze the orthostatic or the astatic actions, respectively, and, in particular, one may determine the nature of the astatic parameters. Afterward, the heterostatic action may be investigated by the formulas for combination which were indicated in Part IV, particularly by Formulas (74), (75), (97), (101), and (102). The investigation, except in special cases, will not proceed further than the determination of these differential equations. As to their solution in detail, and their application to individual cases, reference is made to the several specific treatments which are listed in the Bibliography (Part VII).

29.—Straight Columns.‡—Cases in which the axial load is applied continuously throughout the length of the column, or in which there is a continuous transverse elastic support, have been analyzed by Timoshenko, Greenhill, and Zimmerman.§ Such columns will not be included in the present analysis. The discussion will be confined to cases in which the axial load is transferred at a finite number of definite points and in which the transverse support is at definite points. In such cases, the column may be divided into sectors, each of which is between two consecutive points at which there is a transverse reaction or a change of the axial load. Fig. 9 shows such a sector. The distances, x, along the column are measured from the lower end of this sector. The x-axis may be fixed in space or attached in some way to points of the center line of the column. The deflections, y, are measured from the x-axis, positive toward the right. The axial load, P, is the astatic component of the load. P may be inclined relative to the x-axis, as shown in Fig. 9 (a), but, in this case, it may always be replaced, as indicated in Fig. 9 (b), by a load, P, along the x-axis combined with a horizontal force, X, and a couple, Y, at the lower end.

* See Bibliography, Part VII, Section *A*.

† See the works by Timoshenko previously mentioned, or, A. Ostenfeld, "Teknisk Elasticitetslære", 3d Edition (1916), p. 442.

‡ Bibliography, Part VII, Section *C*.

§ Bibliography, Part VII, Sections *A* and *C*.

X and Y may be considered as constants which are to be determined by the end conditions. Their positive directions are shown in Fig. 9 (b). The remainder of the notation follows:

Let M = bending moment due to the orthostatic or transverse load alone. M is considered as positive when causing compression to the left, tension to the right.

M' = bending moment under the combined influence of the transverse and the axial load, considered positive in the same direction as M.

EI = stiffness factor (modulus of elasticity times moment of inertia of the cross-section). EI may vary as a function of x.

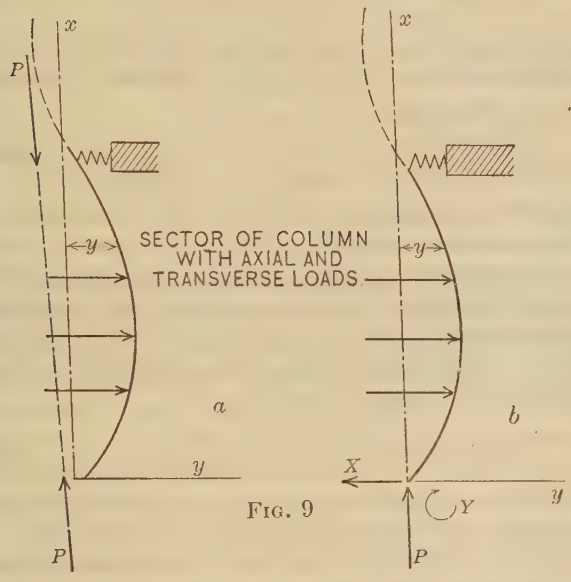


FIG. 9

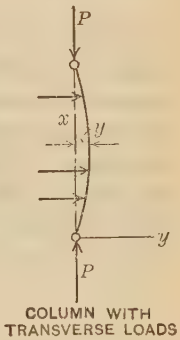


FIG. 10

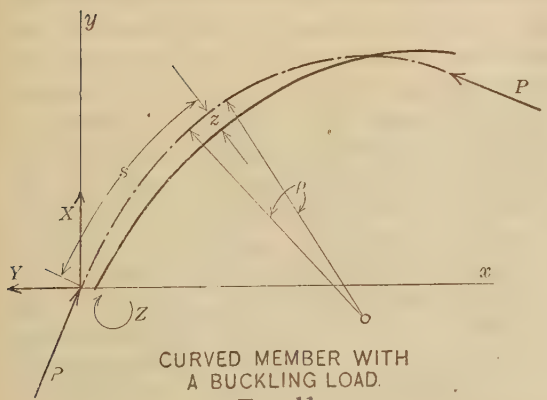


FIG. 11

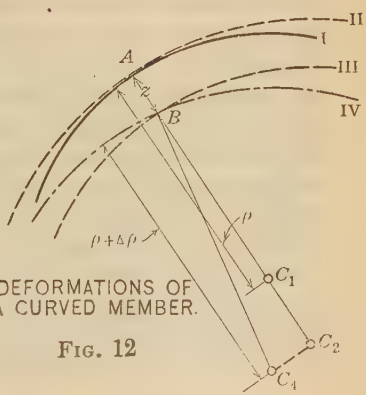


FIG. 12

With positive directions as indicated, the equation of flexure is,

$$E\,I\,\frac{d^2\,y}{d\,x^2} = -\,M' \qquad (136)$$

From Fig. 9 (*b*), it follows that:

$$M' = M + Py + Xx + Y \qquad (137)$$

therefore, the differential equation applying to the sector is:

$$E\,I\,\frac{d^2\,y}{d\,x^2} + Py + Xx + Y = -\,M \qquad (138)$$

where the constants, X and Y, are to be chosen so as to satisfy the end conditions. The end conditions are defined by the character of the supports, by the methods of transfer of the axial load, or by the connection with adjoining sectors above and below.

$P = X = Y = 0$ gives orthostatic action, which may be analyzed by the common methods of technical mechanics. $M = 0$, $EI =$ constant, P positive (compression) gives solutions of the type:

$$y = C_1 \sin x\sqrt{\frac{P}{E\,I}} + C_2 \cos x\sqrt{\frac{P}{E\,I}} - \frac{Xx + Y}{P} \qquad (139)$$

where C_1 and C_2 as X and Y are constants. $M = 0$, $EI =$ constant, P negative (representing the case of tension) gives solutions of a form similar to Formula (139), but with hyperbolic sine and cosine instead of the sine and the cosine. When both ends of the sector are hinged, and the load is applied, as in Fig. 10, then these particular end conditions give $X = Y = 0$ in Formula (138). When $M = 0$, $EI =$ constant, the solution, Formula (139), applies, but is in this case reduced to the well-known form:

$$y = C_1 \sin x\sqrt{\frac{P}{E\,I}} \qquad (140)$$

Formulas (139) and (140) define the particular astatic actions dealt with in Part II. Formula (139) applies also to such cases of eccentric action, or action with end couples, as were treated in Article 9. The formula leads directly to Formula (15) for the bending moment produced in these actions.

*30.—Curved Members.**—Fig. 11 shows a curved elastic member. The dotted curve indicates the undeflected center line, and the full curve the deflected center line. Both curves are assumed to be contained in the xy-plane. The origin is at one end of the undeflected curve. We shall begin by considering the special case in which astatic action is produced by a load system, P, consisting of the following forces: First, two equal forces, P, tangent to the undeflected center line, are acting at the ends toward the structure so as to produce mainly compression. Second, a distributed normal pressure, p, is applied on the convex side of the curve; the distribution of p is such that the undeflected center line is a funicular curve (line of pressure) for those pressures; the intensity of the pressures, p, varies in such a way proportional to the end forces, P, that the end forces hold the distributed pressure in equilibrium. Third, at the lower end (at the origin), there is, in addition to

* See Bibliography, Part VII, Section *G*.

the concentrated force, P, an end load which may be resolved into the three components X, Y, and Z. The two forces, X and Y, act through the origin in the directions $+y$ and $-x$, respectively (as indicated by the arrows in the diagram); and the couple, Z, acts in a clockwise direction. X, Y, and Z may be considered as reactions produced by the remainder of the total astatic load, P, and are zero when $P = 0$. Fourth, at the upper end there is, in addition to the concentrated force, P, a load similar to that of the load, X, Y, Z, at the lower end. All these loads are parts of the astatic component, P, of the total load. In the general case of heterostatic action, there is, in addition, an orthostatic load-component, W.

It is further denoted that:

s = the distance measured along the curve from the origin.

z = the deflection normal to the curve, positive toward the right when the observer looks in the direction of increasing s.

ρ = the radius of curvature, measured positive from the curve in the positive direction of z.

M, M', and EI = the same as in Article 29. The moments are considered as positive when they cause compressions to the left when the observer looks in the direction of increasing s.

The orthostatic load may produce not only bending moments, M, but also axial pressures. The latter will be assumed to be small compared with the astatic load, P. Then, it may be assumed, with a sufficient approximation as far as the stresses and deflections are concerned, that the axial pressure in the heterostatic action is the same as in the astatic action produced by P. This procedure is equivalent to considering the orthostatic load as replaced by an equivalent load which produces the same bending moment, M, but no axial pressures.

It is assumed that the center line is not a very sharp curve, so that the radius of curvature at any point is several times the transverse dimensions of the structural member. Then the equation of flexure may be written:

$$\frac{1}{\rho + \Delta\rho} - \frac{1}{\rho} = -\frac{M'}{EI} \qquad (141)$$

The left-hand expression may be transcribed as follows:* In Fig. 12, Curve I is an element of the undeflected curve, while Curve IV represents the same curve element in the deflected position. The transformation of the element, Curve I, into the element, Curve IV, may take place through the intermediate positions, Curves II and III, which are defined as follows: Curve II is tangent to Curve I at the point A, which is under consideration; Curve III has the same center of curvature, C_2, as Curve II, and the same radius of curvature as Curve IV. The centers of curvature of the four elements in Fig. 12 are C_1, C_2, C_2, and C_4, respectively. Let z_2 denote the deflection of Curve II relative to Curve I, then the increase of curvature

* Another derivation based on the formula for the radius of curvature in polar co-ordinates is given in A. Ostenfeld's "Teknisk Elasticitetslære", 3d ed. (1916), p. 480.

(curvature = reciprocal of radius of curvature) at the transformation, Curve I to Curve II, is expressed as $\frac{d^2 z_2}{ds^2}$. At the transformation, Curve II to Curve III, the increase of curvature is $\frac{1}{BC_2} - \frac{1}{AC_2}$, but when C_1 C_2, and the deflection $AB = z$, are small distances, this difference is very nearly equal to, and may be replaced by, the quantity, $\frac{1}{BC_1} - \frac{1}{AC_1} = \frac{z}{\rho^2}$. The transformation, Curve III to Curve IV, does not change the curvature any further. Since $\frac{d^2 z}{ds^2} = \frac{d^2 z_2}{ds^2}$, it follows that the total increase of curvature, or the left side in Formula (141), is $\frac{d^2 z}{ds^2} + \frac{z}{\rho^2}$. Hence, the equation of flexure can be written as follows:

$$EI\left(\frac{d^2 z}{ds^2} + \frac{z}{\rho^2}\right) = -M' \quad \ldots\ldots\ldots\ldots\ldots\ldots (142)$$

Consideration of Fig. 11 gives in the heterostatic action,

$$M' = M + Pz + Xx + Yy + Z,$$

and this expression, substituted in Formula (142), gives the equation of flexure:

$$EI\left(\frac{d^2 z}{ds^2} + \frac{z}{\rho^2}\right) + Pz + Xx + Yy + Z = -M' \quad \ldots\ldots (143)$$

The corresponding Formula (138), applying to straight columns, represents the special case of Formula (143), in which $\rho = \infty$.

With ρ = constant, EI = constant, $M = 0$, and with certain end conditions, the solution of Formula (143) is a trigonometric function, sine or cosine of angles proportional to s. Such solutions define the astatic actions of circular cylinders carrying a uniform outside pressure.

In deriving the differential equation, Formula (143), it was assumed that the pressure, p, was at all places perpendicular to the curve, s. Under such circumstances, the axial pressure in the astatic action is the same in all cross-sections, and equal to the end pressure, P. However, an astatic action is also possible, in which the axial pressure, N, varies. The variation from the value, P, which is the value of N at the point, $s = 0$, is caused by tangential components of the distributed pressure, p. The general condition of pure astatic action is that the undeflected center line is a pressure line for the force set, P, p. The axial forces, N, appear as rays in the corresponding force polygon (or force curve). It follows that N at each point varies proportionally to P, so that P may still be used as a measure of the intensity of the astatic load. When the force set, P, p, can be assumed to remain in a constant position after the flexure, the differential equation will be the same as Formula (143), except that N is substituted for P. As in the special case, the constants, X, Y, and Z, as well as the integration constants must be determined from the end conditions.

*31.—Slabs.**—Fig. 13 shows a part of the slab in its undeflected position. The xy-plane is the central plane of the undeflected slab. The y-axis is assumed to be vertical and the x-axis to be horizontal. The astatic load consists of the following uniformly distributed forces, all parallel to the xy-plane and acting in the xy-plane as long as the slab is undeflected:

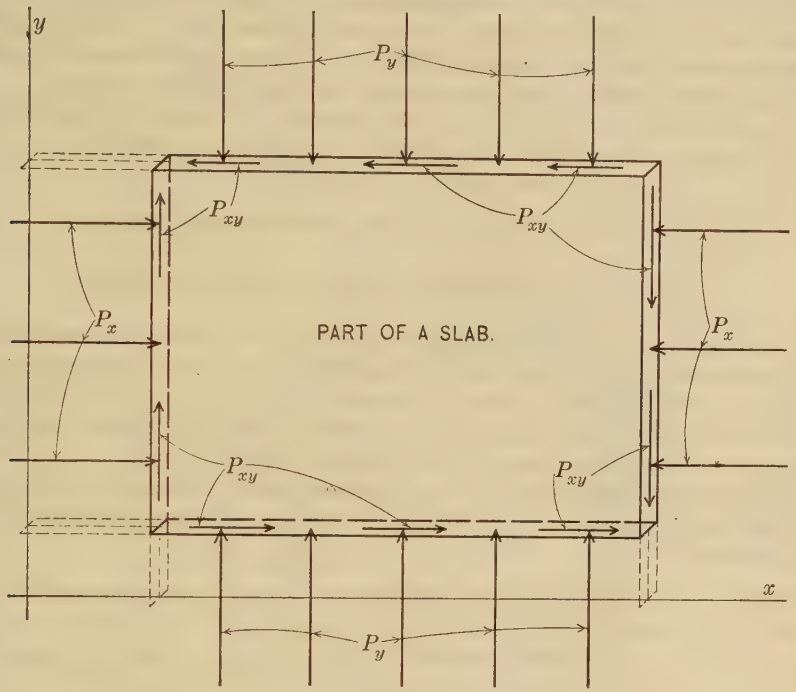


FIG. 13.

Let P_x = the horizontal end pressure in the xy-plane per unit vertical distance.

P_y = the vertical end pressure in the xy-plane per unit horizontal distance.

P_{xy} = the distributed force per unit length along the edges: Horizontal force along the horizontal edges, vertical force along the vertical edges. These horizontal and vertical forces are equal per unit length on account of the law of "shearing forces in pairs".

The orthostatic load is:

w = the surface pressure perpendicular to the slab per unit area. In the general case, w is a function of x and y.

* See Bibliography, Part VII, Section *J*. The derivation of the equation of flexure of slabs in rectangular co-ordinates with loads normal to the surface is given in several treatises, see, for example, A. Föppl, "Technische Mechanik", v. 5, 1907 ed., pp. 97 *et seq.* Concerning the terms in the flexure equation, which are due to the end pressures, see, for example, A. E. H. Love, "Theory of Elasticity" (1906), pp. 528-529. A flexure equation of general form, which includes the effects of end pressures and end shears, is given in B. de Saint-Venant's annotated translation of Clebsch's "Theory of Elasticity", Paris, 1883, Note by Saint-Venant, p. 704, Equation (e_1).

The pressures, P_x and P_y, give uniform horizontal and vertical compressions in the undeflected slab, while P_{xy} produces a uniform horizontal and vertical shear. It is assumed that this uniform distribution will still exist after the slab has deflected, and this assumption is generally warranted as long as the deflections are small compared with the thickness of the slab. The conclusion is that P_x, P_y, and P_{xy} will be transmitted as constant internal pressures and shears parallel to the xy-plane. They may be interpreted as being located centrally in the slab, only they must then be considered as combined with other internal forces, namely, such as represent the usual bending stresses.

Fig. 14 shows the forces acting on the elements, $dx\ dy$, after it has deflected out of its original position. The deflections are defined as follows:

z = the deflection of lower left-hand corner, Point x, y;

$\frac{\partial z}{\partial x} dx$ = increase of deflection from left face to right face; and

$\frac{\partial z}{\partial y} dy$ = increase of deflection from lower face to upper face.

The loads shown acting on the element in Fig. 14 may be listed as follows: The surface load, $w\ dx\ dy$, in the direction, z; the central internal forces parallel to the xy-plane, namely, $P_x dy$ and $P_{xy} dx$, in the x-direction, and $P_y dx$ and $P_{xy} dy$ in the y-direction; in addition, internal shears in the directions, $\pm z$, and bending moments and torsional moments, as given in Table 4. The unit torsional moments, Z, in the horizontal and vertical faces are equal on account of the law of equality of shears in sections perpendicular to one another.

TABLE 4.—Forces and Couples Acting on the Slab Element in Fig. 14.

	Shear in directions, $\pm z$.	Bending moments.	Torsional moments.
Left face	$V_x\ dy$	$X\ dy$	$Z\ dy$
(Direction)	$(-z)$	(xz)	(yz)
Right face	$\left(V_x + \frac{\partial V_x}{\partial x} dx\right) dy$	$\left(X + \frac{\partial X}{\partial x} dx\right) dy$	$\left(Z + \frac{\partial Z}{\partial x} dx\right) dy$
(Direction)	$(+z)$	(zx)	(zy)
Lower face	$V_y\ dx$	$Y\ dx$	$Z\ dx$
(Direction)	$(-z)$	(yz)	(xz)
Upper face	$\left(V_y + \frac{\partial V_y}{\partial y}\right) dx$	$\left(Y + \frac{\partial Y}{\partial y} dy\right) dx$	$\left(Z + \frac{\partial Z}{\partial y} dy\right) dx$
(Direction)	$(+z)$	(zy)	(zx)

By equating the sum of the forces in the z-direction to zero, and dividing by $dx\ dy$, the following condition is found:

$$\frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} + w = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (144)$$

By equating the sum of the moments about a vertical line through the center of the element to zero, and dividing by $dx\,dy$, we find:

$$\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial y} = V_x + P_x \frac{\partial z}{\partial x} + P_{xy} \frac{\partial z}{\partial y} \quad \ldots\ldots\ldots\ldots (145)$$

In the same way, by using a moment axis through the center, parallel to the x-axis, we find:

$$\frac{\partial Y}{\partial y} + \frac{\partial Z}{\partial x} = V_y + P_y \frac{\partial z}{\partial y} + P_{xy} \frac{\partial z}{\partial x} \quad \ldots\ldots\ldots\ldots (146)$$

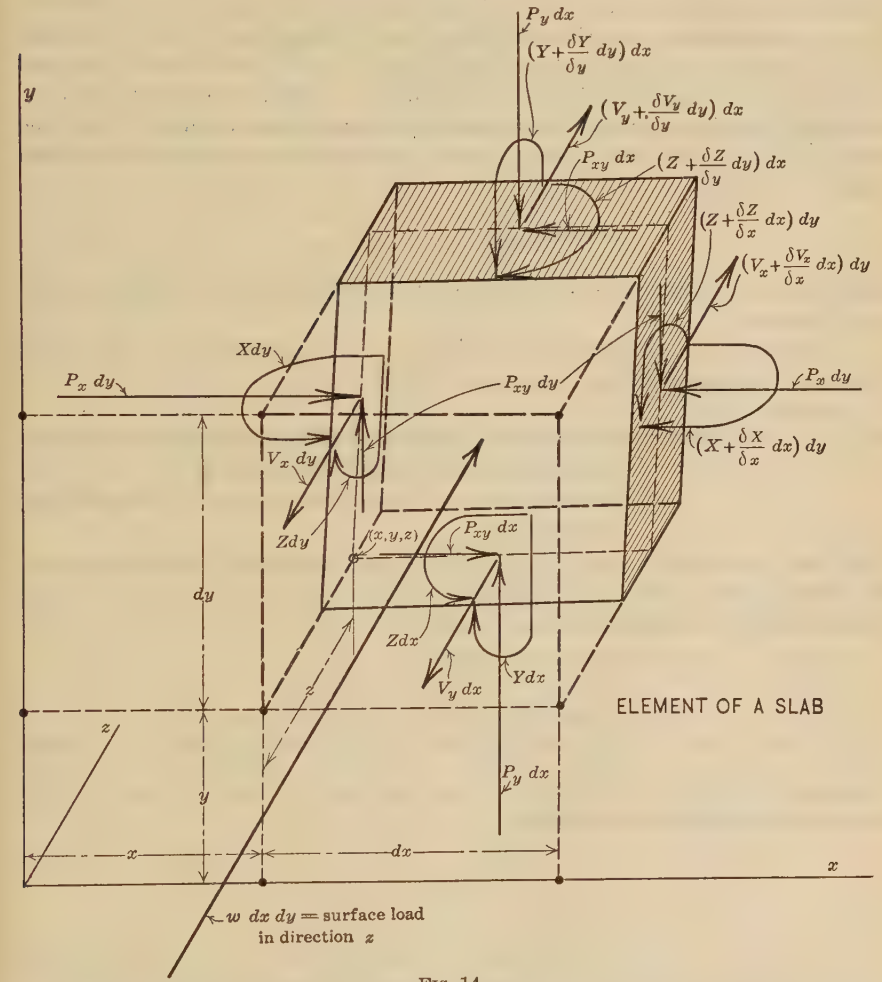


FIG. 14.

By differentiation of Formula (145) with respect to x, and Formula (146) with respect to y, and by adding and combining with Formula (144), we find:

$$\frac{\partial^2 X}{\partial x^2} + 2\,\frac{\partial^2 Z}{\partial x\,\partial y} + \frac{\partial^2 Y}{\partial y^2}$$

$$= -w + P_x \frac{\partial^2 z}{\partial x^2} + 2\,P_{xy} \frac{\partial^2 z}{\partial x\,\partial y} + P_y \frac{\partial^2 z}{\partial y^2} \quad \ldots\ldots\ldots (147)$$

Formula (147) is a general equation of equilibrium on which theory dealing with specific cases may be based. It applies to continuous slabs whether or not they are homogeneous, and whether they are isotropic, that is, having the same properties in different directions, or æolotropic, having different properties in different directions. An example of the latter is the two-way, reinforced concrete slab, in which one may have to assume different stiffnesses in the various directions. Formula (147) applies also to the case of the double system of closely spaced beams for which buckling formulas were given in Article 11. These formulas may be derived from Formula (147) by putting $Z = 0$, as corresponding to the condition of zero torsional resistance, by putting $P_{xy} = 0$ and $w = 0$, and by substituting terms for X and Y, which are proportional to the curvatures in the two directions. Then, values of P_x and P_y, for which solutions different from zero exist, are critical values. When the x-axis and the y-axis are along two edges, such solutions have the following form:

$$z = u_{mn} \sin \frac{m \pi x}{a} \sin \frac{m \pi y}{b} \ldots\ldots\ldots\ldots\ldots\ldots (148)$$

where u_{mn} is a constant which can be used as an astatic parameter, while m and n are integers.

A particularly important case is that of a homogeneous slab, for which buckling formulas were indicated in Article 10.

Assume that

EI = the modulus of elasticity times the moment of inertia of cross-section per unit length; and

K = Poisson's ratio of lateral contraction to longitudinal elongation.

Then, the usual slab theory (see, for instance, the previously quoted treatment in Föppl's work) gives the following relation between deformations and moments in the homogeneous slab:

$$\frac{\partial^4 z}{\partial x^4} + 2 \frac{\partial^4 z}{\partial x^2 \partial y^2} + \frac{\partial^4 z}{\partial y^4} = - \frac{1 - K^2}{E I} \left(\frac{\partial^2 X}{\partial x^2} + 2 \frac{\partial^2 Z}{\partial x \partial y} + \frac{\partial^2 Y}{\partial y^2} \right) \ldots\ldots\ldots (149)$$

Combination with Formula (147) gives then the formula for heterostatic action in homogeneous slabs, namely:

$$\frac{E I}{1 - K^2} \left(\frac{\partial^4 z}{\partial x^4} + 2 \frac{\partial^4 z}{\partial x^2 \partial y^2} + \frac{\partial^4 z}{\partial y^4} \right) + P_x \frac{\partial^2 z}{\partial x^2} + 2 P_{xy} \frac{\partial^2 z}{\partial x \partial y} + P_y \frac{\partial^2 z}{\partial y^2} = w \ldots\ldots\ldots\ldots\ldots (150)$$

As an example, consider the rectangular slab treated in Article 10, which is simply supported along straight edges and has the spans, a and b. It is assumed that no shearing forces like P_{xy} are acting. With $w = 0$, $P_{xy} = 0$, and with co-ordinate axes along two edges, Formula (150) will be satisfied by solutions of the form, Formula (148), on the condition that P_x and P_y take such special, mutually dependent values, Q_a and Q_b, as are indicated by

Formula (21) in Article 10. These are the critical values which produce astatic actions.

The writer will indicate how the bending moment at the center of a simply supported rectangular slab may be found by integration of Formula (150). He will first consider the orthostatic action produced by a uniform surface load, w, acting alone. The computations are simplified by assuming that:

$$b = \pi \quad = \text{short span};$$

$$a = \frac{\pi}{\alpha} = \text{long span};$$

$$w = \frac{1}{\pi^2} = \text{surface load per unit area.}$$

These values give $wb^2 = 1$. The use of a as the ratio of the spans is in accordance with the notation in Article 10, and the results found with these special values may be made to apply in the more general case by multiplication by the proper factors. For example, wb^2, with the notation used in Article 10, is the factor used in expressing the bending moments per unit width.

The x-axis and the y-axis are taken parallel to the long and to the short span, respectively. The origin is at the center, and the surface load, w, may be expressed in a double Fourier series, as follows:

$$w = \frac{16}{\pi^4} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}}{m\,n} \cos m\,\alpha\,x \,.\, \sin n\,y \quad = \frac{1}{\pi^2} \quad \ldots(151)$$

This expression is substituted in Formula (150). In the orthostatic action, we have $P_x = P_y = P_{xy} = 0$. Then, we find the following solution of Formula (150):

$$z = \frac{16\,(1-K^2)}{\pi^4\,E\,I} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}}{m\,n\,(\alpha^2\,m^2+n^2)^2} \cos m\,\alpha\,x \,.\, \cos n\,y \quad \ldots(152)$$

In indicating formulas for bending moments in orthostatic action in Article 10, Poisson's ratio, K, was assumed to be equal to zero. Reasons for deriving formulas on this basis, even when K is known to differ from zero, were stated in Article 10. With $K = 0$, the bending moments are directly proportional to the curvatures. The following expression is then found for the bending moment in the short span:

$$Y = -E\,I\,\frac{\delta^2 z}{\delta\,y^2}$$

$$= \frac{16}{\pi^4} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}\,n}{m\,(\alpha^2\,m^2+n^2)^2} \cos m\,\alpha\,x \,.\, \cos n\,y \quad \ldots\ldots(153)$$

By introducing the factor, wb^2, on the right side, Formula (153) can be made to apply to any spans and unit load. Then, by substituting $x = y = 0$, which corresponds to the center of the slab, the following expression is found for the bending moment in the short span at the center:

$$M_b = \frac{16\,w\,b^2}{\pi^4} \sum_{1,3..}^{m} \sum_{1,3...}^{n} \frac{-(-1)^{\frac{m+n}{2}}\,n}{m\,(\alpha^2\,m^2+n^2)^2} \quad \ldots\ldots\ldots(154)$$

This double infinite series defines a function of wb^2 and a. It can be shown that the function, Formula (154), is rather well approximated by the simplified Formula (28), which was indicated in Article 10.

Each separate term in the double infinite series, Formulas (152), (153), and (154), is seen to represent a case of pure astatic equilibrium as produced by the end loads, P_x and P_y, alone (compare Formula (148)), that is, these series are of the type represented in Formula (99) which may be written in this case, as follows:

$$F = \Sigma F^{(n)} \quad \ldots\ldots (155)$$

where F and $F^{(n)}$ are effects in orthostatic and astatic actions, respectively. Formula (155) may also be interpreted as a form of equation, Formula (60), which is the corresponding formula applying in the simplified case which was treated in Article 17. Formula (155) is also of the same form as Formula (11) in Article 7, which expresses the moments in columns. In accordance with Formula (102), the corresponding effect in heterostatic action is expressed, as follows:

$$F' = F + \Sigma \frac{P}{Q_n - P} F^{(n)} \quad \ldots\ldots (156)$$

Formula (156) may be interpreted as a case of Formula (75). It is of a form similar to Formula (12). In applying Formulas (155) and (156) to the present case of flexure, the index, n, should be replaced by a double index such as m, n. The writer will confine himself to the cases in which $m = n = 1$ gives the smallest critical value, Q, of the astatic load, P. The result is that Formula (156), with the separate terms in Formula (154) substituted for $F^{(n)}$, is a series which begins to converge rapidly immediately after the term corresponding to $m = n = 1$. When the higher terms in the summation in Formula (156) are omitted, that is, when only the first term, which corresponds to $m = n = 1$, is included, the following approximate value is found for the term which must be added to the moment, M_b, in Formula (154) when the end loads, P, are introduced in addition to the already existing surface load, w:

$$M_b'' = \frac{16}{\pi^4} \frac{w\, b^2}{(\alpha^2 + 1)^2} \frac{P}{Q - P} \quad \ldots\ldots (157)$$

Formula (157) is the same as Formula (30) in Article 10.

The case of the double system of crossing-beams for which formulas were given in Article 11, is analyzed in a similar way. The orthostatic flexure due to the distributed load, w, alone depends on the differential equation:

$$\frac{\delta^4 z}{\delta x^4} + \frac{\delta^4 z}{\delta y^4} = \frac{w}{EI} \quad \ldots\ldots (158)$$

which can be integrated by the same method as that used in integrating Formula (150), in the case of the rectangular slab. By means of a trigonometric series similar to Formulas (151), (152), and (153), the following value is found for the moment at the center:

$$M = \frac{16\, w\, b^2}{\pi^4} \sum_{1,3\ldots}^{m} \sum_{1,3\ldots}^{n} \frac{-(-1)^{\frac{m+n}{2}} m}{n\,(m^4 + n^4)} = 0.077\, wb^2 \ldots (159)$$

The value, $0.077\ wb^2$, was indicated by Formula (36) in Article 11. The first term in Formula (159) is as follows:

$$\frac{8}{\pi^4}\ wb^2 = 1.07 \times 0.077\ wb^2 \qquad (160)$$

It is assumed again that the lowest critical astatic load, Q, corresponds to $m = n = 1$. By substituting Formula (160) for the first term, $F^{(n)}$, in the summation in Formula (156), and omitting the remainder of the terms which, in the case considered, may be assumed to be very small, the following approximate expression is found for the moment at the center in the heterostatic action which is caused by the combined influence of the end loads, P, and the surface load, w:

$$M' = 0.077\ wb^2 \left(1 + 1.07\ \frac{P}{Q - P}\right) \qquad (161)$$

Formula (161) is the same as Formula (37) in Article 11.

VI.—Summary.

32.—The investigation deals with structural actions in which the stresses are not proportional to the loads, although the proportional limit of the material has not been exceeded and the deflections remain small. A number of examples of such actions have been mentioned in the Introduction, in Article 1. A simple example is that of the buckling of a slender column under an axial load. A central axial load acting alone on a straight homogeneous column produces "astatic" action; a transverse load acting alone would produce "orthostatic" action; while a combination of the two loads produces a "heterostatic" action. The three terms—orthostatic, astatic, and heterostatic—as applying to structural actions in general, were introduced in Article 1, where orthostatic action is characterized by proportionality of the stresses to the loads; astatic action by the neutral elastic equilibria; and heterostatic action by the combined action of loads which separately would produce orthostatic or astatic action. In the Bibliography (Part VII), a great number of previous investigations of astatic actions are listed.

The present investigation deals with astatic action, and, in particular, with heterostatic action. The study of the latter was made possible by the introduction of a set of "astatic parameters" (Article 15), which were found to have peculiar properties, in particular that of "orthogonality". The general theory is given in Part IV. The use of the astatic parameters leads to simple and general solutions of otherwise complex and difficult problems. The main results of the theory are expressed in Formulas (59), (109), (74), and (97), for the parameters, and Formulas (75), (101), and (102), for the effects in general in heterostatic action.

Although the general theory makes use of the principle of least action, which is expressed in energy equations, certain specific cases of the simpler kind are solved more easily by the direct statical principles which are expressed in differential equations of equilibrium. In Part V, differential equations are set up, which represent cases of straight columns (Formula (138)), curved members (Formula (143)), and slabs (Formulas (147) and

(150)). Special solutions of these equations lead to the specific cases for which formulas are given in Parts II and III.

Part II deals with columns carrying axial and transverse loads at the same time. The solutions depend on the general Formula (12), which is a special case of the Formulas (75), (101), and (102) which are derived in Part IV. The results are found in terms of infinite series, such as Formula (7), but these series appear to be rapidly convergent in the cases investigated, and they can be replaced by approximate formulas with only a few terms in each. Such approximate formulas are given in Table 1, Article 4, and are so simple that they may be used readily by designers of structures. Numerical and graphical results are given in Article 6. In Part III, formulas are given for buckling of slabs and systems of crossing-beams. These cases, also, are special cases to which the results of the general analysis in Part IV may be applied directly.

VII.—Classified Bibliography.

The Bibliography which follows, does not attempt to include references dealing exclusively with straight columns which have constant cross-sections and which carry axial end loads only. This case is treated more or less extensively in almost every textbook on Mechanics of Materials. It is sufficient for the present purpose to point to Euler's analysis of columns, which dates back to 1757, and to the progress which has been marked by the introduction of such formulas as those of Rankine, the parabolic, and the straight-line formulas. A great deal of experimental work has been completed in this field. The deviations from Euler's formula are usually explained by the inevitable presence of small initial eccentricities. In fact, some of the texts* interpret the formulas mentioned as approximate representations of the secant formula which applies to eccentric loading, and which may be used when some definite "equivalent initial eccentricity" has been introduced.

One work on straight columns, of a comparatively recent date, is mentioned in the Bibliography. It is the investigation of columns by Kármán (Section *B*). Kármán's experiments and analysis are noteworthy, due to the light which they throw on the influence of stresses above the proportional limit. This matter is important with reference to the shorter columns. Among the other works mentioned in the Bibliography, particular attention is called to S. Timoshenko's treatment of a great variety of cases of elastic buckling (Section *A*), and also to the analysis of cylindrical shells by Goupil (Section *G*), in which a special case of the general formula for heterostatic action is derived. It is in the nature of the matter that the following Bibliography cannot lay any claim to completeness.

Section *A*.

This section includes discussions dealing with a variety of cases of buckling or with cases of a general nature.

BRYAN, G. H. On the Stability of Elastic Systems. Cambridge, *Proceedings,* Philosophical Soc. (1888), pp. 199-210.

FÖPPL, A. and L. Drang und Zwang, v. 2 (1920), pp. 315-390.

* See, for instance, J. E. Boyd, "Strength of Materials," 1917 Edition.

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KÁRMÁN, TH. von. Festigkeitsprobleme im Maschinenbau. Encycl. der Math. Wiss., IV, 27 (1910). See in particular pp. 370-385.

LOVE, A. E. H. Mathematical Theory of Elasticity. Second Edition. Cambridge, 1906. See Index, p. 550, under "Stability".

LORENZ, H. Technische Elastizitätslehre. 1913, pp. 314-365.

OSTENFELD, A. Teknisk Elasticitetslære. Third Edition. Copenhagen, 1916, Chapters V and VII.

RODE. Beitrag zur Theorie der Knickerscheinungen. *Eisenbau* (1916), pp. 121-131, 210-218, 299-305.

——Ueber Schwingungen. *Eisenbau* (1917), pp. 250-258.

SOUTHWELL, R. V. On the General Theory of Elastic Stability. *Philosophical Transactions,* Royal Soc., Lond. (A), v. 213 (1913), pp. 187-244.

TIMOSHENKO, S. Sur la Stabilité des Systèmes Elastiques. *Annales des Ponts et Chaussées* (1913), III, pp. 496-566; IV, pp. 73-132; and V, pp. 372-412.

——Einige Stabilitätsprobleme der Elastizitätstheorie. *Zeitschrift für Math. u. Phys.,* v. 58 (1910), pp. 337-385.

——Also, works in Russian (Yearbooks of the Polytechnical Institutes of Petrograd and Kiev, 1906, etc.).

Section *B*.

This section includes references to column action or buckling in which stresses beyond the proportional limit are considered.

BRIDGEMAN, P. W. The Collapse of Thick Cylinders under High Hydrostatic Pressure. *Physical Review* (1912), pp. 1-24.

KÁRMÁN, TH. von. Untersuchungen über Knickfestigkeit *Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens,* v. 81 (1910), 44 pp.

SOUTHWELL, R. V. The Strength of Struts. *Engineering,* v. 94 (1912), pp. 249-250.

Section *C*.

This section includes the more general cases of straight columns, characterized as follows: The axial loads may be transferred at other points than at the ends; or, the cross-section may vary; or, there may be transverse supports at intermediate points; the supports may be rigid or elastic, concentrated, or distributed.

BAIRSTOW, L., and E. W STEDMAN. Critical Loads for Long Struts of Varying Section. *Engineering,* v. 98 (1914), pp. 403-404.

BLASIUS, H. Träger kleinster Durchbiegung und Stäbe grösster Knickfestigkeit bei gegebenem Materialverbrauch. *Zeitschrift für Math. u. Phys.,* v. 62 (1914), pp. 182-197.

BOYD, J. E. Investigation of the Compressive Strength of Spruce Struts, etc. *Technologic Papers,* U. S. Bureau of Standards, No. 152 (1920), 43 pp.

CASE, J. An Approximate Graphical Treatment of Some Strut Problems. *Engineering,* v. 106 (1918), pp. 699-700.

ELWITZ. Die Knickkraft in Stäben mit sprungweise veränderlichem Trägheitsmoment. *Zentralblatt der Bauverwaltung* (1917), pp. 516-520, 525-528.

ENGESSER, FR. Ueber die Knickfestigkeit von Stäben veränderliches Trägheitsmomentes. *Zeitschrift* des Oesterr. Ingenieur- u. Architekten-Vereines (1909), pp. 544-548.

GREENHILL,* A. G. Determination of the Greatest Height Consistent with Stability That a Vertical Pole or Mast Can Be Made, and of the Greatest Height to Which a Tree of Given Proportions Can Grow. Cambridge, *Proceedings,* Philosophical Soc., v. 4 (1881), pp. 65-73.

INOKUTY, A. Column of Uniform Strength. *Journal,* Coll. of Eng., Imperial Univ. of Tokio, v. 5, No. 8 (1914), pp. 223-244.

KAYSER, H. Beziehungen zwischen Druckfestigkeit und Biegungsfestigkeit. *Zeitschrift* des Ver. deutscher Ingenieure (1917), pp. 92-97, 124-127.

ONO, A. On the Stability of Long Struts of Variable Section. *Memoirs,* Coll. of Eng., Kyushu Imperial Univ. v. 1, No. 5 (1919), pp. 395-406.

VIANELLO, LUIGI. Graphische Untersuchung der Knickfestigkeit gerader Stäbe. *Zeitschrift* des Ver. deutscher Ingenieure (1898), pp. 1436-1443.

WALLACE, WM. Theory of Taper Columns. *Engineering,* v. 94 (1912), pp. 831-832.

ZIMMERMAN, H. Der gerade Stab mit stetiger, elastischer Stützung und beliebig gerichteten Einzellasten. *Sitzungsberichte* der k. preussischen Akad. der Wissenschaften (1905), pp. 898-912.

——Der gerade Stab auf elastischen Einzelstützen. *Ibid* (1907), pp. 235-244.

——Die Knickfestigkeit des geraden Stabes mit mehreren Feldern. *Ibid* (1909), pp. 180-212.

* An error of computation in Greenhill's paper is corrected by Timoshenko, *Annales des Ponts et Chaussées* (1913), III, p. 514.

Section *D*.

In this section references are given to buckling in which twisting couples are active: Bending of the center line of a shaft into a spiral shape by twisting couples at the ends or by such couples combined with compression; tipping or bending out sidewise of a high narrow beam loaded by forces which are initially in the plane of maximum rigidity (plane of symmetry containing the largest dimension of the cross-section).

GREENHILL, A. G. On the Strength of Shafting when Exposed Both to Torsion and to End Thrust. *Proceedings,* Inst. of Mech. Engineers (1883), pp. 182-209.

PRANDTL, L. Kipp-Erscheinungen (Thesis in Munich). 1899. 75 pp.

Section *E*.

References are made herein to columns or compression members built up as trusses or frames.

AMERICAN SOCIETY OF CIVIL ENGINEERS. Final Report of Special Committee on Steel Columns and Struts. *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919-20), pp. 1583-1688.

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ENGESSER, FR. Ueber die Bestimmung der Knickfestigkeit gegliederter Stäbe. *Zeitschrift* des Oesterr. Ing. u. Arch. Ver. (1913), pp. 769-772.

GÉRARD, GUSTAVE L. Théorie physique de la résistance des pièces comprimées à treillis. *Revue Universelle des Mines et de la Métallurgie* (1913) (3), pp. 178-203, 221-246.

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MÜLLER-BRESLAU, H. Ueber exzentrisch gedrückte gegliederte Stäbe. *Sitzungsberichte* der k. preussischen Akademie der Wissenschaften (1910), pp. 166-181.

Section *F*.

Side stiffness of trusses and arches, particularly the side stiffness of bridges of the pony-truss type are referred to in this section.

ENGESSER, FR. Die Zusatzkräfte und Nebenspannungen eiserner Fachwerkbrücken. v. 2 (1893), pp. 105-157.

KEELHOFF. La stabilité des membrures comprimées des ponts métalliques. *Annales des Ponts et Chaussées* (1920), II, pp. 193-231.

MÜLLER-BRESLAU, H. Graphische Statik, II, 2.

OSTENFELD, A. Teknisk Statik II. Second Edition. Copenhagen, 1913, pp. 483-492.

——Aabne Broers Sidestivhed. *Ingeniören,* Copenhagen (1914), pp. 475 *et seq.;* (1918), pp. 543-549, 577-582.

——Die Seitensteifigkeit offener Brücken, Beton und Eisen. 1916, pp. 117-120, 147-148.

YASSINSKY. Works in Russian, see the previously quoted Memoir by Timoshenko. *Annales des Ponts et Chaussées* (1913).

ZIMMERMAN, H. Die Knickfestigkeit der Druckgurte offener Brücken. *Zeitschrift für Bauwesen* (1910), pp. 329 *et seq.;* also, an enlarged reprint, Berlin (1910), 53 pp.

——Der Stabeck auf elastischen Einzelstützen. *Sitzungsberichte* der k. preussischen Akad. d. Wiss. (1907), pp. 326-338; also, the works of Zimmerman mentioned under Section *C*.

* The empirical formula proposed by Gérard covers the cases of shorter as well as longer spans and panel lengths. The formula indicated by Lossier gives values which agree rather well with results of Timoshenko's analysis. The latter deals primarily with the cases of long spans. It is maintained by Mathieu that certain tests indicate Gérard's formula as sufficiently safe, and Lossier's formula as in some cases too much on the safe side when the spans are not particularly large.

Section *G*.

This section includes references to circular ring-resisting forces in its plane, particularly a uniform outside pressure, and cylinders resisting forces perpendicular to their axes, particularly a uniform outside pressure.

BELSETSKY, STANISLAUS. De la stabilité d'équilibre dans un cas particulier de pièce courbe. *Comptes Rendus*, v. 156 (1913), pp. 1056-1058.

BRESSE, J. A. CH. Cours de mécanique appliquée. Paris, 1859, pp. 326 *et seq.*

BRYAN, G. H. Applications of the Energy Test to the Collapse of a Long Thin Pipe under External Pressure. Cambridge, *Proceedings*, Philosophical Soc. (1888), pp. 287-292.

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FORCHHEIMER, PH. Zur Festigkeit weiter Rohre. *Zeitschrift* d. Oesterr. Ing. u. Arch. Vereines (1904), pp. 133-137, 149-152.

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*Gives formula applying to pipes with stiffening rings.

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DISCUSSION

GEORGE PAASWELL,* M. AM. SOC. C. E. (by letter).—For the first time there is presented before the Society a rational precis of column action, unfortunately, by a mathematical physicist, using the word, unfortunately, in the sense that the language he speaks is foreign to most of his readers. The importance of the subject, as well as of the matter presented, justifies an attempt to annotate the contents of the paper, both as a matter of criticism and as a matter of elucidation to readers interested in column theory.

Fundamentally, the author classifies all column action into three, more or less characteristic, actions: The true buckling—where flexure is the principal action—the author terms "astatic equilibrium"; where distortion is directly proportional to the load, as in the short column, or in ordinary transverse flexure, the related equilibrium is termed "orthostatic equilibrium"; and a mixed condition containing both types of equilibrium is termed "heterostatic equilibrium." Summarized briefly in other terms: Orthostatic, in which $\frac{l}{r}$ is less than 20; heterostatic, in which $\frac{l}{r}$ lies between 20 and 300; and astatic, in which $\frac{l}{r}$ is greater than 300. The values of $\frac{l}{r}$ are, of course, approximate, but are those generally used to distinguish the column types.

Taking up the work presented by the author in the logical order of development, the basis of the analysis is the energy function, the resilience factors as found in a stressed column or thin rod, as the case may be. The development of the work equations may have been predicated on the energy function of Lord Kelvin, with probably more facility in comprehension by engineering readers, but the methods used are undoubtedly better and are the basis of the most modern research in mathematical elasticity.

The voluminous data presented in forming the equations may be necessary from the standpoint of a mathematical physicist—as far as establishing the requisite requirements of rigor—but they are confusing to the engineer. Briefly, the configuration of the elastic curve of columns, slender rods, or thin plates, plane or curved, is determined by a set of parameters, u_m, etc. If the shape of the elastic curve is given by an expression, $y = u \sin a x$, then, any variation in the parameter, u, affects the amplitude of y, but not its phase, exhibiting one character of the parameter.

The equations for work are known to engineers in the form of Castigliano's theorems—the theory of least work and its corollaries following from the well known principle of least action of D'Alembert. Although his theorem should apply essentially to conservative systems,† yet the results may be extended to systems which are not conservative. Astatic systems probably may be regarded as such.

* Cons. Engr., New York City.

† See, for example, Byerly, "Generalized Co-ordinates".

The final useful form of the parameter, as given by Formula (74), follows from the minimum property, and the "effect" which may here be termed the moment, is built up of terms of Formula (74), obtaining the author's fundamental equation. The form as given in Formula (125) involves the introduction of primary eccentricities and, as the author points out, is the basis of most existing formulas.

Proceeding from Formula (74), the formulas of Table 1, and those in Formulas (7) to (10), are established. This should be noted, however. Formula (13) is predicated on very small displacements. The exact expression takes the form:

$$M_n = C_n \, c\,n\,a\,(x + K)$$

where $c\ n$ is the modular cosine, an elliptic function, and characterizes the phenomena grouped under the general term, elastica. For small values, cn becomes a cosine, and $K = \frac{\pi}{2}$, giving the sine form as previously noted. The form, as given, does not lend itself to development in the Fourier series. The higher forms of buckling given by the larger values of n imply necessarily finite displacements, so that the trigonometric series would be in serious error. The modular functions with the values of K adjusted to have the periods fixed by the values of n, gives the more correct forms of the elastica. However, as the coefficients in the Fourier series form a rapidly converging series, the higher terms play no rôle in the analysis, and the equations presented give substantially reliable results.

If one concedes that it is possible to give rational column formulas to cover so-called heterostatic action, the author has completely covered the field of column formulation. The writer will endeavor to show that it is not feasible to establish a formula which will rationally exhibit column action, but that, nevertheless, a mode of procedure does exist, which permits a rational mode of column design.

It may be well at this point to review briefly previous attempts to analyze column action and to see what results have been obtained.

Euler pointed out in 1778 that vertical columns do not break under vertical loads by mere crushing, but that flexure of the column will be found to precede rupture. He proceeded to develop the equations which determine the maximum load to hold a column at a given deflection. Lagrange expressed the limiting load equations in a similar fashion. Robinson* points out that experimental research fails to check Euler and Lagrange. In discussing the results of Hodgkinson's tests, Pearson suggests that there may be some value in the Eulerian hypothesis when it is properly modified so as to include the effect of compression. It is interesting to note here that the analysis of the author shows that when compression is allowed for, the form of the equations in heterostatic action is not changed.

* "A System of Machanical Philosophy", 1805.

E. Lamarle* notes that up to a certain ratio of length to diameter, the maximum load ought to be calculated from the formula:

$$P_0 = \frac{E F S_0}{n}$$

where S_0 is the greatest stretch that can be given without passing the elastic limit; F is the area of the section; and n is Poisson's ratio; the ratio is denoted by K in the paper. Beyond this ratio, the maximum load is given by the Euler expression. It is noted that these expressions seem to reconcile the Euler theory with the existing experimental data.

Rankine proposed the expression bearing his name. He states that this expression was first suggested by Tredgold and afterward revived by Gordon who determined the values of the constant from Hodgkinson's experiments.

A very theoretical study of the action of slender rods is given by Clebsch in his "Theory of Elasticity." The equation of the elastic curve of the rod is determined from the solution of the differential equation:

$$E I \frac{d^2 \phi}{d s^2} = C \sin \phi$$

ϕ is the angle which the tangent to the curve at any point a distance, s, from the terminus makes with any fixed direction. The solution of this equation involves the elliptic functions. An exhaustive analysis of this curve, covering all the higher systems of stability—the higher criteria of astatic equilibrium—has been given by A. E. H. Love, in "A Treatise on the Mathematical Theory of Elasticity," in which an interesting analogy (Kirchoff's analogy) with the motion of a pendulum is illustrated. The question of the elastic stability of systems is also discussed in the text, but, as the rods are slender ones, the treatment is practically that of Euler.

The introduction of the linear law by the late T. H. Johnson, M. Am. Soc. C. E., and the parabolic expression of the late J. B. Johnson, M. Am. Soc. C. E., need not be discussed here, except possibly to quote an excerpt from the paper of Mr. T. H. Johnson,† as follows:

"Before closing this article, it will be well to revert to the older formulas and the reasons why they fail to give satisfactory results * * *. Gordon fell into error, not only from paucity of experiments at his command, but also by following Tredgold's assumption that a certain portion of the area is required to resist flexure. Under this assumption the modulus of compression would remain as a factor in the equation for all length ratios, but with diminishing influence, becoming zero only when $\frac{l}{r}$ becomes infinite: whereas it now appears that the modulus of compression ceases to be a factor when $P = \frac{K}{3}$ [K is the Euler factor] * * *."

In discussing a paper‡ by the late A. J. DuBois, M. Am. Soc. C. E., Mr. T. H. Johnson has hinted at a possible solution of the column theory using

* "Memoire sur la Flexion du Bois", *Annales* des Travaux Publics de Belgique, IV, pp. 1–46 (1846).

† "On the Strength of Columns", *Transactions*, Am. Soc. C. E., Vol. XV (1886), p. 517

‡ "A New Formula for the Strength of Columns", *Transactions*, Am. Soc. C. E., Vol. XXVII (1892), p. 69.

the energy developed as a working basis. This, it is noted, has been accomplished by the author, utilizing the results of modern research, as given in the Bibliography (Part VII).

The rapid growth of column theories and their vague connection with the increasing volume of test statistics made it necessary to draw a distinction between the column of theory and that of practice. Not a sharp or clearly defined distinction, but one sufficient to limit the theory and still provide an opening for empiric adjustment. William Cain, M. Am. Soc. C. E., defines this theoretic column as the "ideal column", and proceeds to effect its theoretic study.* His definition of the ideal column is: "prismatic homogeneous columns, having the force, *P*, applied at one end, in the direction of the axis or line through the centers of gravity of the cross-sections."

He points out a vital fact in the analysis of the column as carried out by Euler and one which has not been clearly enunciated before, namely, that Euler's formula gives the load at which bending just begins. In this connection, Mr. W. E. Lilley, an earnest and brilliant student of the column, both as a mathematical and as an engineering problem, has demonstrated that Euler's formula considers bending only and ignores the effect of the direct load as a compressive force on the transverse sections.† This, of course, is quite clear from an analysis of the expressions given by the author.

Amplifying the theory of the ideal column, Anson Marston, M. Am. Soc. C. E., has extended the analysis to cover the case of a load of known eccentricity, obtaining the so-called secant expression for the stress.

The converse problem of the "practical column" has been discussed‡ by J. M. Moncrieff, M. Am. Soc. C. E. Intricate expressions were obtained which Professor Cain reduced to the standard secant form.

The results of experiments are always somewhat disappointing. There are, of course, numerous uncertain elements which influence the strength of a column, such as type of form, details, processes of manufacture, fabrication, etc., all of which may not be of calculable nature. The influence of any or all of these elements may be obviated by a wise selection of forms; but this is rarely possible. New tests bring with them new problems to be solved by further experimentation. The ideal function of tests is to supply certain physical constants, which, when substituted in formulas mathematically derived, determine at once the strength factors of a given type of column. Practically, tests fall far short of performing this function and are usually applied as determining the usable stresses in a column of given material and proportions. This is a restricted field for tests, but to the present time they have not justified their extension to broader fields. The late A. P. Boller, M. Am. Soc. C. E., added a discouraging note: "In my opinion no formula will ever give satisfactory results, and dependence must be placed upon experimental charts."

* "Theory of the Ideal Column", *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898) p. 96.

† *Engineering*, July 2d, 1909.

‡ "The Practical Column under Central or Eccentric Loads," *Transactions*, Am. Soc. C. E., Vol. XLV (1901), p. 334.

It is seen that not much progress has been made in the development of column formulas, and this paper may well be taken as a last word in the attempt to formulate column action. At this point, one may well ask "Is a rational column formula possible?"

Briefly, a column is an engineering structure subjected to a compressive force of a determinate character and to a flexure absolutely indeterminate and unpredictive with any mathematical certainty. This, of course, refers to columns presumably axially loaded. The introduction of flexural stresses occurs in a manner which can only form a matter of conjecture. The piece may not be straight due to fabrication, shipping, or erection methods. The loading may not be applied at the geometric axis of the column due to the manner of the connections or other details of the structure of which the column forms a member. The column may be detailed so that the individual shapes comprising the column entity do not act as a unit, so that the distortions under load are not uniform (this is not to be confused with astatic action or equilibrium); or it may be a heavily rolled shape the metal of which does not possess uniform elastic properties.

In a number of cases, where there is a transverse load, or where known moments are introduced at the column ends, it is possible to predict the flexure with reasonable accuracy. For the present, the standard column type—an independent structural unit—is under discussion. Tests serve a general purpose in giving clues to column action, but like theory itself, they must be interpreted with a clear apperception of the physical improbability of defining column action rigorously.

Emphasizing the fact that despite elaborate analysis, there is no way of determining the flexure moment of what may be termed simple heterostatic action, that is, no transverse loading, it seems inevitable that one must answer the leading question, "Is a rational column formula possible?" in the negative. Since some method of analysis is necessary, the proper mode of procedure would seem to be, to find out under what conditions, referring, of course, to unique cases, the value of the moment, M, can be made determinate.

Stated in slightly different language, the problem for each individual case examined seems to be to learn just what is the true position of the applied load. This eccentricity would lead to the selection of the proper formula, as given by Professor Westergaard, or to some of the simplifications now in vogue. Again, in a number of cases, it is possible to determine just what moment is introduced into the column by its adjoining members. If such moment is calculable, it is the moment to be used in the general formula.

The column is usually treated as a separate and distinct unit taking its load, presumably axial, but no other stress, from its neighboring units. This is the ideal, but far from the actual, condition. Every structural member, whether beam, column, or only a connecting detail, is a part of a system of framework, and it is through the distortion of the frame by various types of loading that the several members, comprising the frame, are stressed. By a study of the frame itself, therefore, it may be possible to make determinate the factor, M, in the column formula. A few examples may illustrate the possibilities of this mode of analysis.

The simple bent, as shown in Fig. 15, is a common type of construction, and usually the girder is assumed to rest lightly on the column, that is, it is designed as a simple beam with free supports. On the other hand, the designer may purposely introduce such details as will fix the corner, making it rigid, thus transferring a moment into both the column and the girder. It is possible, of course, that in the actual case such details will be used as will develop a joint intermediate between the free and the fixed ones, the designer assuming a simply supported beam and a column taking no moment from the joint.

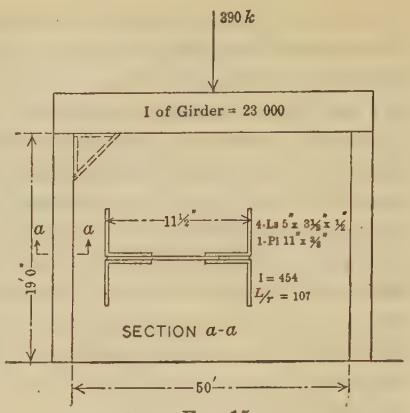

FIG. 15.

It is important to note that the joint, as a simple connection, places no restraint on the deflection of the girder and the girder must throw a substantial, if not the entire, portion of its load on the inner edge of the column. One may safely assume that the entire load will thus be placed. The eccentricity of the load is thus measurable and M can be determined. Assign arithmetical values to the members shown in Fig. 15. The value of the moment is $185 \times 0.48 = 89$ ft-kips. Following the method ordinarily used and obtaining the permissible stress from the building code formula of New York City,

$$\left(p = 15\,200 - 58\,\frac{L}{r}\right)$$

the allowable unit stress is 9 000 lb. per sq. in., which the area and slenderness ratio of the column just satisfy. Using the formula:

$$F = \frac{P}{f} + \frac{M_c}{I}$$

and taking an allowable stress for f of 16 kips per sq. in., and with the other functions of the section as noted in the diagram, solving for M, there is found a value of 45 ft-kips., only half the moment as determined from the eccentric location method. Although it is true that one may say that the entire concentration of the load on the column edge is only a matter of conjecture, it is a possible and, more emphatically, a probable condition and gives a calculable premise which is a vital point in rational and consistent design.

Take an alternative condition: The joint is made a fixed one, thus bringing the girder reaction concentric with the column section. Assume that the girder is loaded, as shown in Fig. 15, with a single concentration at its midpoint. Computing the moments from *Bulletin No. 108,* University of Illinois, "Analysis of Statistically Indeterminate Structures by the Slope Deflection Method," by Messrs. Wilson, Richart, and Weiss, on the assumption that the detail fixing the joint is dimensioned so that it reinforces the column section to a point 2 ft. below the girder, it is found that the column section must

be such as to carry a moment of 160 ft-kips. Clearly, it is not enough merely to call a joint fixed. It must be made so, and the column must be designed to meet the moment. On the other hand, if a detail is introduced which, in effect, fixes the connection, care must be taken to see that the column can safely take the moment induced. The writer is aware that this question is amply covered in most treatises on secondary stresses, but it is quite doubtful whether, in ordinary structural design of simple frames, the effect of the joint is properly examined. As a matter of comment, however, it seems safe to say, that, in this case, also, it is possible to compute a value of M and that resort to a column formula is unnecessary.

Several other cases might be mentioned, such as building columns, framing between girders, viaduct columns, and columns which are subjected to a definite transverse load, such as subway columns and the like. Many more types will undoubtedly occur to the reader. It is sufficient to state that, in engineering practice, in the majority of cases, it is possible to find a calculable moment in the column member.

The column formula is predicated on an uncertain flexure, while it has been noted that it is possible to observe the moment to which the ordinary column is subjected. It seems, therefore, a rational method of design to make provision for the stresses which can be directly computed, using such permissible unit stresses as tests on short compression blocks exhibit, or tension tests, since these give about the same result as the compression tests. It may be objected that this method of design ignores the true column action induced by the accidents of fabrication or erection. This objection, although a valid one, may be applied to all structural units. If correct analysis has been used in developing a section to fit into a given structure, with proper provision for flexure, where such flexure can be calculated, the accidental stresses must be assumed to be taken care of in the factor of safety.

This method demands great alertness on the part of the designer. The deformations of the entire structure of which the column forms only one unit must be carefully studied. The standard column formula is a blanket allowance for all such uncertain loadings. The rational method suggested gives the designer no such royal path to a simple design. It is a postulate of good engineering design that the details should fully develop the structural members which they interconnect. The converse of this may now well be emphasized. Structural members should be designed to carry safely the stresses which the details bring to them. Too often this is not done. There is too much child-like faith in "axial loads", "free supports", etc. It is true that in monumental structures a correct apperception of the conditions is had, and the details and the main sections are designed with consistency; but this is not so in structures of every-day design. Joints are made fixed by heavy details to secure a rigid structure, while the individual members are treated as separate and distinct units.

Two conditions may form an exception to the suggested method of design. Cases may occur where, as far as a thorough study can determine, a column is axially loaded with no outside induced flexure. As this is the type of column which is usually tested, it may be safe to say that column formulas

predicated on tests of similarly formed columns are rational ones to use. The results of Professor Westergaard's investigations indicate that the Rankine expression is well founded*. In this connection, it may not be amiss to state that a series of tests on framework would give more profitable results than tests on columns alone.

The second exception is an important one. Throughout the author's analysis, and in engineering design in general, the metal is assumed to have uniform elastic properties. The heavy rolled sections do not possess such uniformity, and low ultimate and low useful limit points are found in columns assembled with heavy shapes. The use of metal of this character will give a section of low average resistance in comparison with a section of the same area, but composed of lighter shapes. For this condition, a study of the report of the Special Committee on Steel Columns and Struts† will indicate just what should be used in the way of safe stresses. This exception, where there is a way of calculating a flexural stress, merely affects the total permissible stress, the proper formula to be used being in no way affected by the fact that the ultimate stress is lower than that found for thin rolled sections.

One may say in conclusion that, although it does not seem possible to discover a rational column formula, a rational method of column design does exist, but that a more detailed study of the structure is required than is ordinarily given. This objection, however, cannot be considered a libel against the method suggested. In the choosing of a column section, only half the problem is solved. The column details are vital, possibly more vital than the selection of the section members, as disastrous failures have indicated. A rational mode of design permits a rational mode of detailing. A well-knit consistently designed and detailed column contains a far greater factor of safety, even though the accidental stresses usually attributable to so-called column action have been neglected, than the column designed in accordance with some of the numerous formulas, with the vague rule-of-thumb methods of proportioning the details.

Eugene E. Halmos.‡ M. Am. Soc. C. E. (by letter).—This paper is probably the most thorough treatise that the writer has seen on the theory of combined buckling and "cross-bending" of columns and slabs. The general theory applicable to other structural forms, so interestingly demonstrated, will also be appreciated by all students of mechanics of structures. The author is to be commended for indicating and illustrating the extreme usefulness of one of the best tools of higher mathematics, the Fourier series, in the solution of problems encountered in engineering design.

It is gratifying to note that the author has proven the formulas in general use for designing columns (Formulas (1), (2*a*), and (2*b*)) to be on the safe side

* See, also, "A Review of Column Formulas for Bridge Design", *Engineering News-Record*, March 10th, 1921, p. 431, etc. The suggested formula is:

$$p = \frac{16\,000}{1 + \frac{L^2}{18\,500\,r^2}}.$$

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

‡ Designing Engr., Parsons, Klapp, Brinckerhoff and Douglas, New York City.

by a considerable margin, except in the case of fixed end columns and distributed side loads (Case IV, Formula (5)). However, as designers will dimension the columns on the basis of maximum transverse moment, which, in the case of a fixed beam, occurs at the ends, it will be seen, by comparison with the author's Formula (6), that the use of the "approximate" formulas results in safe structures.

All the author's examples refer to the particular cases where "orthostatic" loading increases the effect of "astatic" action. There is, however, a case, often met in practice, where the opposite is true, namely, an upright thin cylinder or drum supporting a vertical load and being filled with a gas or liquid exerting a greater pressure on the wall than that of the surrounding air or liquid. The writer proposes to show the derivation of formulas for such a "heterostatic" loading. A slightly modified presentation of Professor Forchheimer's solution, published in a little known pamphlet,* will be given. First, the equilibrium conditions of a cylindrical drum supporting only a vertical load will be derived, and then the influence of inside and outside pressures will be demonstrated.

A.—Buckling of an Upright Thin Cylinder Under a Load Distributed Along the Rim.—Quoting the author, "a thin cylindrical shell or tubular strut may buckle into a double-curved surface under the influence of a compressive load parallel to the axis" (page 580). This form of failure will be treated by the writer, not the assymetrical buckling investigated by R. Lorenz in his paper referred to in the author's Bibliography, Part VII, Section *H*.

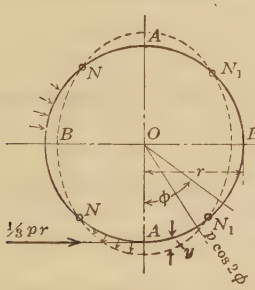

FIG. 16.

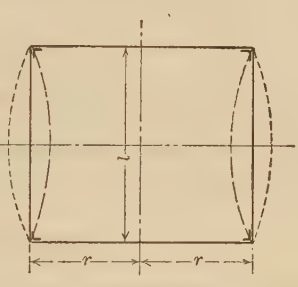

FIG. 17.

In Fig. 16, let the circle drawn in full line represent the horizontal section of a vertical drum stiffened on top and bottom, the radius, r, being assumed to be large compared with the thickness of the shell, and let the dotted elliptical line be the same section after the drum has deformed under the influence of a compressive load distributed uniformly over the rim of the cylinder. Fig. 17 is a vertical section of the shell, the full lines representing the unstressed wall of a perfect cylinder, the dotted lines the deformed wall along Diameter *A-A*, and the dashed lines the deformed wall along Diameter *B-B*. If it is assumed that the shell consists of a large number of independent vertical strips, it is evident that, should a horizontal thin band be welded around the drum, this band would be pushed radially outward between the nodal points, $N\ N'$, and radially inward between the points, $N\ N$, and $N'\ N'$, by

* "The Berechnung Ebener und Gekrümmter Behälterböden", 1909.

forces proportional to the distance of the elliptical contour from the original circular one (Fig. 16).

Consider a ring of unit height, sectional moment of inertia, I, modulus of elasticity, E, subjected to forces of the character described. Denoting with p the (unknown) maximum pressure intensity, the unit pressure at any point of the ring can then be expressed by,

$$p \cos 2\phi = p\ (2\cos^2\phi - 1)$$

in which ϕ is the angle between the vector and the radius, $O\ A$.

At A, where $\phi = 0$, we have a reaction:

$$p\,r \int_{-\frac{\pi}{2}}^{0} (2\cos^2\phi - 1)\sin\phi\, d\phi = p\,r \left[\frac{-2\cos^3\phi}{3} + \cos\phi\right]_{-\frac{\pi}{2}}^{0} = \frac{p\,r}{3} \quad ..(162)$$

and a moment, M_0, yet unknown.

At the point where $\phi = \phi_1$, the moment of all pressures between 0 and ϕ_1, will be:

$$\int_0^{\phi_1} p\,r\,(2\cos^2\phi - 1)\,r\sin(\phi_1 - \phi)\,d\phi^*$$

$$= p\,r^2 \sin\phi_1 \int_0^{\phi_1} (1 - 2\sin^2\phi)\cos\phi\,d\phi - p\,r^2\cos\phi_1 \int_0^{\phi_1} (2\cos^2\phi - 1)\sin\phi\,d\phi$$

$$= p\,r^2 \sin\phi_1 (\sin\phi_1 - \frac{2}{3}\sin^3\phi_1) + p\,r^2\cos\phi_1 \left(\frac{2}{3}\cos^3\phi_1 - \cos\phi_1 + \frac{1}{3}\right)$$

$$= \frac{p\,r^2}{3}(-2\cos^2\phi_1 + \cos\phi_1 + 1)$$

or, dropping the subscript,

$$\frac{p\,r^2}{3}(\cos\phi - \cos 2\phi)$$

to which the moment of the reaction, $\frac{1}{3}\,p\,r$, and M_0 should be added. The total moment, therefore, at the point, ϕ, will be:

$$M = M_0 + \frac{1}{3}\,p\,r^2\,(1 - \cos\phi) + \frac{p\,r^2}{3}(\cos\phi - \cos 2\phi)$$

$$= M_0 + \frac{2}{3}\,p\,r^2\sin^2\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (163)$$

According to Boussinesq,† the deformation of a ring generally follows the law:

$$y + \frac{d^2y}{d\phi^2} = \frac{M\,r^2}{I\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (164)$$

Substituting the value of M from Formula (163), we have the differential equation:

$$\frac{I\,E}{r^2}\left(y + \frac{d^2y}{d\phi^2}\right) = M_0 + \frac{2}{3}\,p\,r^2\sin^2\phi$$

the integral of which is:

$$\frac{I\,E}{r^2}\,y = M_0 + \frac{2}{9}\,p\,r^2\,(1 + \cos^2\phi) + C_1\sin\phi + C_2\cos\phi$$

---

* The change in the length of r, due to the deformation of the shell, is neglected. This is permissible because this deformation, compared with the radius (which is assumed to be large), is very small before failure occurs. Consequently, the forces, $p\cos 2\phi$ are also small.

† *Comptes Rendus*, Vol. 97 (1883) p. 843.

in which C_1 and C_2 are the constants of integration.

By hypothesis (Fig. 16):

$$\frac{I\,E}{r^2}\frac{d\,y}{d\,\phi} = -\frac{4}{9}\,p\,r^2 \sin\phi\cos\phi + C_1\cos\phi - C_2\sin\phi$$

must vanish for both $\phi = 0$ and $\phi = \frac{\pi}{2}$, making $C_1 = C_2 = 0$, and,

$$\frac{I\,E}{r^2}\,y = M_0 + \frac{2}{9}\,p\,r^2\,(1 + \cos^2\phi) \ldots\ldots\ldots\ldots\ldots\ldots (165)$$

M_0 can now be solved by applying the condition that the length of the circumference of the ring remains unchanged when the shell is deformed, that is,

$$\frac{I\,E}{r^2}\int_0^{\frac{\pi}{2}} y\,d\,\phi = \left[M_0\,\phi + \frac{1}{9}\,p\,r^2\,(3\,\phi + \sin\phi\cos\phi)\right]_0^{\frac{\pi}{2}} = 0$$

$$M_0 = -\frac{1}{3}\,p\,r^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (166)$$

which value, substituted in Formula (165), results in

$$y = \frac{p\,r^4}{9\,I\,E}(2\cos^2\phi - 1) = \frac{p\,r^4}{9\,I\,E}\cos 2\,\phi \ldots\ldots\ldots\ldots (167)$$

representing the deformation of a ring radially loaded with forces, $p \cos 2\,\phi$.

Consider, now, a vertical strip of the drum shell, of unit width and length (or height) l, which buckles under a load, Q. Such buckling will be resisted by the horizontal rings with forces proportional to the deformation of the vertical strip. It is assumed that the cylindrical wall of the drum is able to turn freely (in other words, is hinged) at the top and bottom about the stiffening rings. From the theory of buckling of columns, it follows that the strip will deform into a half wave of a sine curve (Fig. 18), and, denoting by q the maximum intensity of the resistance exerted by the rings, the unit pressure resisting the buckling at a height, x, above the base of the strip is given by:

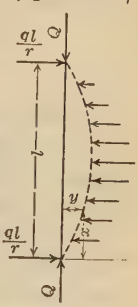

FIG. 18.

$$q \sin\frac{\pi\,x}{l}$$

The reactions at the top and bottom of this strip, corresponding to these horizontal forces, will be:

$$\int_0^{\frac{l}{2}} q \sin\frac{\pi\,x}{l}\,d\,x = \left[-q\,\frac{l}{\pi}\cos\frac{\pi\,x}{l}\right]_0^{\frac{l}{2}} = \frac{q\,l}{\pi} \ldots\ldots\ldots (168)$$

The total moment at the point, x_1:

$$M = Q\,y + \int_0^{x_1}(x_1 - x)\,q \sin\frac{\pi\,x}{l}\,d\,x - \frac{q\,l}{\pi}\,x_1 = Q\,y$$

$$- \left[q\,x_1\,\frac{l}{\pi}\cos\frac{\pi\,x}{l} - q\,\frac{l}{\pi}\,x\cos\frac{\pi\,x}{l} + q\,\frac{l^2}{\pi^2}\sin\frac{\pi\,x}{l}\right]_0^{x_1}$$

$$- q\,\frac{l}{\pi}\,x_1 = Q\,y - q\,\frac{l^2}{\pi^2}\sin\frac{\pi\,x_1}{l} \ldots\ldots\ldots\ldots\ldots (168a)$$

With the notation used, the differential equation of the elastic line becomes,

$$I\,E\,\frac{d^2\,y}{d\,x^2} = -\,Q\,y + q\,\frac{l^2}{\pi^2}\sin\frac{\pi\,x}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (169)$$

the solution of which, considering that y must vanish for both $x = 0$ and $x = l$, can be written as:

$$y = \frac{q\,l^4}{\pi^2\,Q\,l^2 - \pi^4\,I\,E}\sin\frac{\pi\,x}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (170)$$

Now, by hypothesis,

$$p\cos 2\,\phi = q\sin\frac{\pi\,x}{l}$$

and y in Formulas (167) and (170) is identical, therefore,

$$\frac{r^4}{9\,I\,E} = \frac{l^4}{\pi^2\,Q\,l^2 - \pi^4\,I\,E}$$

from which the critical load which can be supported on the unit length of the rim of a thin cylinder of given r, l, I, and E, is:

$$Q = \frac{\pi^2\,I\,E}{l^2}\left(1 + \frac{9\,l^4}{\pi^4\,r^4}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (171)$$

If the thickness of the shell is t, then $I = \frac{t^3}{12}$, and Formula (171) becomes,

$$Q = \frac{t^3\,E}{l^2}\left(0.822 + 0.076\,\frac{l^4}{r^4}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (171a)$$

It will be noted that for $r = \infty$, Formula (171) reverts to the well known Euler formula for columns hinged at the top and bottom. The influence of Poisson's ratio was neglected in the derivation of Formula (171), so that the value of Q is slightly on the safe side.

B.—Buckling of an Upright Thin Cylinder Under a Load Distributed Along the Rim and Supporting a Uniform Radial Inside Pressure.—The ring of unit height previously considered is assumed to support inside radial forces of constant intensity, p_1, the magnitude of which is large compared to $p\cos 2\,\phi$. This inside pressure results in a ring tension, $T = p_1\,r$, tangent to the original circular contour and, therefore, acting with a lever, y, on the displaced elements of the deformed ring, Fig. 19. The moment equation, Formula (163), therefore, changes to:

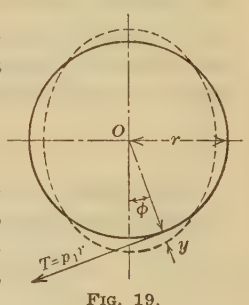

FIG. 19.

$$M = M_0 + \frac{2}{3}\,p\,r^2\sin^2\phi + T\,y \quad \ldots\ldots\ldots\ldots\ldots\ldots (172)$$

and the differential equation of the elastic line to:

$$\frac{I\,E}{r^2}\left(y + \frac{d^2\,y}{d\,\phi^2}\right) = M_0 + \frac{2}{3}\,p\,r^2\sin^2\phi + T\,y \quad \ldots\ldots\ldots\ldots (173)$$

Noting that $\frac{d\,y}{d\,\phi}$ vanishes for $\phi = 0$ and $\phi = \frac{\pi}{2}$, as seen in Fig. 19, the

integral of Formula (173) can be written as:

$$y = \frac{r^2 M_0}{I E - T r^2} + \frac{4}{3} \frac{I E p r^4}{(I E - T r^2)(3 I E + T r^2)} - \frac{2}{3} \frac{p r^4 \sin^2 \phi}{3 I E + T r^2} \quad (174)$$

which can be readily proven by forming $\frac{d^2 y}{d \phi^2}$ and substituting this and the expression for y in the left-hand member of Formula (173).

The condition that the length of the quadrant remains unchanged will give the value of M_0:

$$\frac{I E - T r^2}{r^2} \int_0^{\frac{\pi}{2}} y \, d \phi = M_0 \frac{\pi}{2} + \frac{4}{3} \frac{I E p r^2}{3 I E + T r^2} \frac{\pi}{2} - \frac{2}{3} \frac{I E - T r^2}{3 I E + T r^2} \frac{\pi}{4} p r^2 = 0$$

from which, as before,

$$M_0 = -\frac{1}{3} p r^2$$

and, finally,

$$y = \frac{1 - 2 \sin^2 \phi}{3 (3 I E + T r^2)} p r^4 = \frac{p r^4}{3 (3 I E + T r^2)} \cos 2 \phi \quad \ldots\ldots (175)$$

Formula (175) holds simultaneously with Formula (170),

$$y = \frac{q l^4}{\pi^2 Q l^2 - \pi^4 I E} \sin \frac{\pi x}{l}$$

in which, again, $q \sin \frac{\pi x}{l}$ is identical with the forces, $p \cos 2 \phi$, developed in the ring and resisting distortion; the inside pressure, p_1, is in equilibrium with the tangential tension, T. Therefore, eliminating y between Formulas (170) and (175), and solving for Q, we obtain:

$$Q = \frac{\pi^2 I E}{l^2} \left(1 + \frac{9 l^4}{\pi^4 r^4}\right) + \frac{3}{\pi^2} \frac{l^2}{r^2} T \quad \ldots\ldots (176)$$

indicating, by comparison with Formula (171), the substantial increase in the critical load which can be supported on the rim of a drum when inside pressure is acting.

In terms of the thickness of the wall:

$$Q = \frac{t^3 E}{l^2} \left(0.822 + 0.076 \frac{l^4}{r^4}\right) + 0.304 \frac{l^2}{r^2} T \quad \ldots\ldots (176a)$$

C.—Buckling of an Upright Thin Cylinder Under a Load Distributed Along the Rim and Supporting a Uniform Radial Outside Pressure.—An outside pressure of intensity, p_2, will produce a tangential compressive force, $P = p_2 r$, in the ring. The critical load which can be supported by the drum, by the same reasoning as demonstrated, in Section *B*, can be written as:

$$Q = \frac{\pi^2 I E}{l^2} \left(1 + \frac{9 l^4}{\pi^4 r^4}\right) - \frac{3}{\pi^2} \frac{l^2}{r^2} P \quad \ldots\ldots (177)$$

or, in terms of the thickness of the shell,

$$Q = \frac{t^3 E}{l^2}\left(0.822 + 0.076\,\frac{l^4}{r^4}\right) - 0.304\,\frac{l^2}{r^2}\,P \ldots\ldots\ldots (177a)$$

This formula shows the substantial decrease in the supporting power of a drum or tubular strut when under outside pressure, and it also shows that buckling may occur even though no vertical load is applied, if,

$$P = \frac{\pi^4\, I\, E\, r^2}{3\, l^4}\left(1 + \frac{9\, l^4}{\pi^4\, r^4}\right)$$

or,

$$p_2 = I\, E\left(\frac{3}{r^3} + \frac{\pi^4\, r}{3\, l^4}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (178)$$

and, in terms of the thickness of the shell, if,

$$p_2 = \frac{t^3\, E}{r^3}\left(0.25 + 2.706\,\frac{r^4}{l^4}\right) \ldots\ldots\ldots\ldots\ldots\ldots (178a)$$

A cylinder of length, l, without stiffening rims, must behave in the same manner as one infinitely long with stiffened ends; substituting $l = \infty$ in Formula (178), we get, for the buckling load of cylinders with outside radial pressure,

$$p_2 = \frac{3\, I\, E}{r^3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (179)$$

which simple formula was first given by Boussinesq.*

Edward Godfrey,† M. Am. Soc. C. E. (by letter).—This paper enriches the science of column mathematics with new formulas and new views as well as new terms. The writer believes it is a legitimate criticism of the paper to state that it does not offer much help to the practical designer. Only a few men in the Profession, outside of the colleges, will be able to follow this paper or even to understand and use the formulas derived.

The writer has searched the paper for a formula that would give the theoretical collapsing pressure on a thin tube subjected to external pressure, but could not find it, although the Synopsis holds out the promise that this problem, among others, is solved.

As an example of the problem that faces the designer, take Table 2, in which the relation between the total moment on a column subject to bending to the moment from bending alone, is given. Here, one finds that a certain fraction of the Euler load exerted endwise on a column, will increase by a definite ratio any moment due to lateral loads, regardless of what the moment from lateral load may be. To know this is useful if it has application in a practical design, but what the designer wishes to know is, what end load or what side load may be allowed against a column and still be safe in the design. He also wishes to know to what class of columns Table 2 should be applied.

For example, if a factor of safety of 5 on the Euler load is used in the case of a hinged-end column with a center side load, the moment due to the

* *Comptes Rendus*, Vol. 97 (1883), pp. 844 and 1131.

† Structural Engr. (Robert W. Hunt and Company), Pittsburgh, Pa.

side load will be increased 20 per cent. Will the column be safe? Or, suppose the column is slender and a factor of safety of 1.8 is considered sufficient, the end load will double the moment from the side load, regardless of what that moment may be. European engineers use the Euler formula for designing columns with a factor of safety of 5. In America many long derrick booms are designed, that have an actual factor of safety of 1.5 or 2, and the weight of the boom furnishes the lateral load; or the dead end of the boom line may be attached to the middle of the boom, and the upward force may exceed the boom weight.

Between these two extremes, there must be a medium that represents safe and economic design. The European practice of using the Euler formula with a factor of safety of 5 cannot be defended, because the Euler formula does not give even an approximation of the ultimate strength of permissible structural columns, and a factor of safety of 5 for slender columns, where the Euler load does apply, is entirely too large and results in a waste of material.

As the author points out, there is not a start of failure in a slender column until the Euler load is nearly reached. Why, therefore, should there be such a wide margin between the working load and the ultimate? In permissible structural columns there is actual unit stress which is a good fraction of the actual elastic limit of the metal, therefore, the factor of safety should be of some magnitude to take care of overload and of the so-called fatigue of the metal. In slender columns, the actual unit stress under a factor of safety of 5 may be as little as one-fiftieth of the elastic limit or less. This is one of the cases pointed out by Professor Westergaard, where increased load and increased deflection are not directly proportional. There is a sudden jump to the ultimate.

It is of the utmost importance that designers should be familiar with the following facts concerning column design, namely, the inapplicability of slender column formulas to short columns and the equal inapplicability of formulas for short columns to the design of long columns. The author does not distinguish between short and long columns, nor does he state that the formulas given have an application chiefly, if not solely, to slender members. Increased deflection in slender columns gives rise to increased bending moments due to the endwise load. In admissible structural columns, however, the deflection due to externally applied loads is small.

The author does not recognize nor provide for imperfections in columns. In the writer's judgment, no treatment of the strength of columns can be of definite value, which does not recognize the fact that columns are imperfect, and the imperfections have a great influence on their strength. Perfect columns, perfectly loaded, would have a strength greatly in excess of commercial columns loaded in the manner feasible in practice; this refers to structurally permissible columns and not to slender members. Furthermore, the nature of the imperfection assumed must be possible or one that can be defended.

The Gordon formula for columns is based on the assumed imperfection of the column. The imperfection is a bow or deflection in the column directly proportional to the square of the length and inversely proportional to the depth of the column cross-section, and this imperfection is "frozen" in the

column, that is, increased deflection due to application of the end load is ignored. Let these assumptions be examined. For example, suppose a column is 5 ft. long, has an offset of 0.2 in., or $\frac{1}{300}$ of the length. This column might pass for straight. Now, take a column of the same cross-section, 50 ft. long; to agree with the Gordon formula the deflection would be 100 times as great, or 20 in. This column would be bowed, and the imperfection would be so great that the case would not be classed as simple column action. In this case, what the author points out as being true in some cases, namely, that stress is not proportional to load, is of prime importance.

A coefficient determined by test on a short column, for insertion in the Gordon formula, would give a value based on compressive strength of the steel, and elasticity or buckling of the column as a whole, would have little effect; whereas a coefficient for a slender column would be controlled entirely by the elastic properties of the steel, the properties beyond the elastic limit having no effect whatever. Obviously, no agreement could possibly exist between these two columns that a formula of the Gordon type could reconcile.

A great deal of the column mathematics is useless, because it fails to recognize column facts and deals solely with the theory of the ideal column. Any chart of tests on columns will show that a definite column formula for the ultimate strength of a column is impossible. The wide variation between test results of columns of similar properties is the proof of this.

The writer has published* a paper on the strength of columns, which, in effect, gives a rational derivation of the straight-line formula. The formula derived, although complex, has a locus that coincides almost exactly with the commonly used straight-line formula for ratios permissible in structures. The only assumptions are that the unit stress on the extreme fiber of the column is 16 000 lb. per sq. in. and that the column is imperfect. The imperfection is a bow in the axis of the column, the offset being $\frac{1}{300}$ of the column length. Increased deflection due to endwise load is taken into account, as the author has done in his investigations. This article is re-printed in the writer's book, "Steel Designing", and is believed to be the only attempt to demonstrate the straight-line formula as a rational formula. The author apparently did not consider this paper of sufficient merit to mention it in his Bibliography. It would seem that a demonstration of the rationality of the one formula used generally in America for columns, would be worthy of recognition if only to prove its falsity if it is falsely derived. It is just as important to point out the errors of the literature on the subject as to emphasize the merits.

In the Synopsis of the paper, the fact is recognized that a straight, homogeneous and elastic column will reach a point where it bends out suddenly. This is called the critical value. In the body of the paper, it is shown that this point of sudden bowing is the Euler load. In the Introduction, it is stated:

"The neutral equilibrium, maintained at this constant critical load throughout a continuous range of increasing deflections, is the criterion of pure buckling. The load may be increased beyond the critical value at which the bending ordinarily begins, and still the column may be made to remain in equilibrium in an undeflected state."

* *Railway Age Gazette*, July 2d, 1909.

The condition of a column at the Euler load, the author calls a "stable neutral equilibrium."

With this quotation from the Introduction, and with the idea that a column at the Euler load is in a stable neutral equilibrium, the writer cannot agree. It is of great importance that it be made clear that at the Euler load a column has reached its absolute ultimate load and is not in any kind of equilibrium. That a column is in equilibrium at the Euler load carries with it the idea of reserve strength, but it can be proven by mathematics that the Euler load is the ultimate load that a column can sustain.

The common derivation of the Euler relation finds that for any assumed deflection, the column remaining elastic, there is theoretical equilibrium between the internal resistance and the applied load. From this, it is asserted that the column is capable of sustaining the Euler load at any assumed deflection. The truth is that this is merely the history of the failure of the column. When the Euler load is reached, the column begins to deflect measurably, and continues to deflect without increased load. Nothing could prevent the continuance of this deflection, after it is once started, unless it should be some other means of support for the load, and this is not admissible. There is nothing, therefore, to prevent the failure of the column.

The proof, referred to is this: Assume a column with hinged ends and having a bow with an infinitesimal offset—a straight column, mathematically speaking. Let the curve of this assumed bow be a sine curve. Calculate the endwise load that will double the assumed infinitesimal bow, which load will double the bow an infinite number of times, and is the Euler load. This multiplication of the infinitesimal deflection means a finite deflection and failure. Thus, by an absolute proof, the column is shown to have reached its ultimate load. There is no room here for the conception of equilibrium.

The author asserts, however, that the column may assume a curve made up of a number of half-waves instead of one curve, which would mean that the column is perfectly regular in its imperfection, and would possess a tendency to bow in one direction for a segment of its length and in the other direction in the next segment, and so on throughout its length. The points of inflection would have to be the same distance apart, and that distance, an exact multiple of the column length. The deflected segments would have to deflect the same amount, otherwise, there would not be a balance between the several segments at points of inflection, which would set up unbalanced forces and cause lateral motion. There is nothing in the nature of columns to warrant the assumption of such impossible conditions.

The author states in the quotation given previously that the column may be made to remain in equilibrium in an undeflected state with the load beyond the critical value at which the bending ordinarily begins. If this means that the column may be made to carry more load by supplying lateral support, it is immediately placed in another class, where the formula has no application. In fact, there is no other way to make a column that has reached the Euler load, carry any more load, except by supporting the column laterally, and thus vitiating the formula as regards that column.

The writer finds that few designers have any conception of the strength of slender columns and of the application of the Euler formula to the proportioning of columns. The reason for this is doubtless the character of the common mathematical treatment in books and in papers on columns. Another thing that has a great influence in the same direction is emphasis on the formulas and tests for structurally permissible columns. All of it may be summed up in the failure of writers to differentiate between slender and short columns, the failure to point out clearly the difference between their action under load, and also that in the zone where slender and short columns meet or overlap, the characteristics of each action must be properly considered and combined.

Mathematical writers have been too assiduous in their quest for a perfect column formula. Nothing but a perfect column would approximately measure up to their formula, particularly in a structurally permissible column.

The "whip formula", presented in the writer's book, comes about as near passing through the middle of any group of tests as any curved-line formula in use. As the handle of the whip, or the straight portion, is the only part that concerns structural designers, it is eminently proper that a simple straight-line formula should be used for structural members. An example of how column tests group themselves around a straight line has been given.*

Designers of airplane struts and transmission towers deal with slender columns, and it is important that their characteristics should receive the attention of the Profession. Only recently the writer's attention was called to designs of transmission towers where high unit stresses were used in columns because of the special quality of the steel. Theory demonstrates that in slender compression members, carbon steel has little or no advantage over soft steel, as the modulus of elasticity is the only factor affecting the results. Tests also demonstrate the same thing.

The writer is unable to understand the author's formulas for square slabs supported on four edges. On page 600, the intensity of the moment of such a slab is given as $0.042\ wb^2$. In other places, for example, Formula (36), it is given as $0.077\ wb^2$. Still another value, on page 599, works out as $0.037\ wb^2$ for a square slab.

On page 599, it is stated that in a slab that is square or nearly square, greater moments occur along and across the diagonals at the corners than at the center. How can a slab be subject to bending when and where it is in contact, by hypothesis, with a rigid and straight support? At the corners, the slab is in contact with the supports, and could not bend either in the direction of the diagonal or transverse with it. Surely no one would reinforce a slab against bending in the corner, where there could be no deflection whatsoever and, therefore, no stress on the reinforcement. If any conceivable force could split the slab in the corner, diagonally, or pinch off the corner, what measurable effect would it have on the slab as a whole?

In the Bibliography, the author has neglected to mention some useful and practical tests on the collapsing pressure of tubes subject to external pressure, which tests† were made by Mr. R. T. Stewart, of the University of Pittsburgh.

* Figs. 1, 2 and 5, *Engineering News-Record*, March 10th, 1921, pp. 431-432.

† *Transactions*, Am. Soc. Mech. Engrs., May, 1906.

H. M. Westergaard,* Esq.† (by letter).—The scope of the writer's investigation was defined so as to exclude the cases in which the deflections are large and those in which the stresses exceed the proportional limit. The first limitation excludes the consideration of the very slender structures the deformations of which have developed beyond the initial stage. The second limitation excludes the application of the method to the exact analysis of the heavier structures at the ultimate load, which cannot be reached until the proportional limit has been exceeded. The phases of the subject thus excluded, are essential in a complete study of column action. For example, the action of short or medium-long columns cannot be appreciated fully without taking into account the behavior of the material beyond the proportional limit; concerning this matter, reference is made to the works of Kármán, Southwell, and Salmon.‡

By limiting the field, however, it became possible to develop an analysis which applies to structures of many shapes; and if the inevitable initial eccentricities are taken into account, as in Articles 24 to 26 of the paper, this analysis will explain quite accurately the strength properties, not only of slender, but also of medium slender structures, for example, of medium slender columns. Rankine's formula is shown in this way to apply with rough approximation to the buckling of columns and of certain other structures (see the closing paragraph in Article 26).

Mr. Paaswell has mentioned slender columns with deflections so large that Euler's theory must be replaced by a more general theory of the elastica. This case, strictly speaking, is outside the scope of the investigation, but the results of an exact analysis of it may be used to decide how large the deflections of a column may be before the deviations from Euler's theory become appreciable. A formula given by Schneider,§ as a result of the exact analysis, is quoted: The end load on a hinged-ended column is:

$$P = \frac{\pi^2 E I}{L^2}\left(1 + \frac{1}{8}\left(\frac{\pi}{L}\right)^2 f^2 + \frac{19}{512}\left(\frac{\pi}{L}\right)^4 f^4 + \frac{29}{2048}\left(\frac{\pi}{L}\right)^6 f^6 + \cdots\right)$$

where L is the length measured along the elastic curve (slightly shortened by the axial force) and f is the maximum deflection. When $\frac{f}{L} = \frac{1}{10}$ the load, P, differs from Euler's value only by about 1.25%; that is, with a maximum deflection equal to as much as one-tenth of the length, Euler's theory still gives a close approximation.

In making applications of the analysis to the initial stages of flexure of the very slender structures, the possible influence of the weight of the structure must be carefully considered. For example, according to Euler's formula for hinged-ended columns, a steel blade, 0.025 in. by 1.5 by 36 in. (with $\frac{l}{r} = 5\,000$), would support an end load equal to 0.45 lb., but, as the blade

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‡ For the works of Kármán and Southwell, see Bibliography, Section *B*, p. 651; E. H. Salmon, "Columns" (Oxford Technical Publications, Lond., 1921), 279 pp. This work contains an extensive bibliography and account of previous work.

§ Alois Schneider, "Zur Theorie der Knickfestigkeit", *Zeitschrift* des Oesterr. Ing.- und Arch. Vereines, Vol. 53 (1901), pp. 633–638, 649–653, in particular, p. 638.

weighs 0.38 lb., Euler's theory cannot be applied unless the blade is placed in a horizontal position, on edge. Placed in a vertical position, with simple supports at the ends, the blade would buckle under the influence of its own weight when a small end load is added, but the elastic curve would not be a sine-curve as in Euler's theory; as long as the deflections remain small, there is a neutral or astatic equilibrium at a constant critical end load, but Euler's theory must be replaced by one in which the distributed weight is taken into consideration.

Mr. Paaswell refers to the actions of long, medium-long, and short columns as examples of astatic, heterostatic, and orthostatic actions, respectively. These illustrations are well chosen insofar as Euler's theory may be verified experimentally; for example, with hinge-ended steel columns in which the value of $\frac{l}{r}$ is greater than about 150; and insofar as the inevitable eccentric action of the medium length column may be interpreted, according to Articles 24 and 25, as special cases of heterostatic action; and insofar as the short column with $\frac{l}{r}$ less than about 50, does not buckle until the elastic limit has been exceeded.

Mr. Paaswell discusses a two-legged bent supporting a weight, W, at the center of the girder. A buckling of this bent in its own plane would be classified by the writer as a heterostatic action produced by a heterostatic load; for the load may be resolved into orthostatic and astatic components, as follows: The orthostatic component which, acting alone, produces bending stresses in columns and girder proportional to W, consists of the load, W, at the center of the girder and two upward forces, $\frac{W}{2}$, at the ends of the girder, and the astatic component which, acting alone, produces a neutral equilibrium with indefinite deflections of columns and girder at a critical value of W, consists of two downward forces, $\frac{W}{2}$, at the ends of the girder. The combined effects of the two components of the load may be computed by Formula (75) or Formula (102). Unless the columns are slender, the critical values, Q_n, of W are likely to be so large as compared with the actual value, P, of W, that the terms containing the factors, $\frac{P}{Q_n - P}$, may be omitted in Formulas (75) and (102), and the action may be considered as being orthostatic. A buckling of the bent out of its plane under the influence of W would be an example of the mixed astatic action described in Article 23 if the buckling of the bent in its own plane still remains possible; if not, the buckling out of the plane becomes a pure astatic action, and the load, W, at the center of the girder becomes an astatic load.

The case analyzed by Mr. Halmos, of the vertical cylinder supporting a vertical load and resisting an inside pressure, doubtless, is of interest and importance. The vertical load is an astatic load, because at a critical value it

produces an astatic equilibrium with indefinitely increasing deflections. The writer would classify also the internal pressure as an astatic load; for at a critical negative value of this pressure, that is, at a certain outside pressure, pure buckling is produced. The combined vertical load and inside pressure, then, is an astatic load. The case is analogous to that of a simply supported rectangular slab buckling under the influence of a tension in the direction of one pair of sides and a compression in the direction of the other pair of sides; the diagrams in Fig. 7, in Article 10, show that the presence of tension in one direction produces an increase of the value of the critical pressure in the other direction.

Mr. Godfrey mentions the term, "stable neutral equilibrium", which was used by the writer. The term, perhaps, calls for some explanation. A reference to an equilibrium analogous to that of the column will serve this purpose. Consider a spherical ball resting on a horizontal table: The ball is in a neutral or astatic equilibrium because each position into which it may pass by rolling on the table is a position of equilibrium. This neutral equilibrium may be called stable insofar as there is no tendency for the ball to leave the table. Imagine a horizontal edge placed above the table. The ball may be balanced on this edge; and if it is guided by a pair of vertical boards on both sides of the edge, parallel to the edge, and barely touching the ball, the equilibrium obtained, again, is neutral, because it is maintained throughout a continuous range of positions, and it is stable, because the ball will not tend to leave the edge. If the vertical boards are removed, the equilibrium is still neutral, existing throughout the same range of positions, but it is unstable, for the least tilting force would make the ball fall on the table; ultimately, then, the ball would revert to the original state of neutral equilibrium on the table.

Consider a slender hinged-ended column: If the ends are forced to approach each other by a small definite distance, the column will buckle into a sine-curve with one half-wave; at a small change of this distance, the maximum deflection will change, but the elastic curve remains a sine-curve with one half-wave, and the required end load remains the same, equal to Euler's value; the equilibrium is neutral and stable, and is analogous to the equilibrium of the ball on the table. Let the column be simply supported at the center; it will buckle, then, into a sine-curve with two half-waves; the neutral equilibrium obtained is analogous to that of the ball on the edge between the two boards; the reactions at the central support are of the same nature as the reactions of the boards on the ball. If the central support is removed, the column will still be in a neutral equilibrium in the same range of positions; but a small transverse force will disturb this equilibrium, which, therefore, is unstable, like the equilibrium of the ball on the edge when the boards have been removed. If the ends of the column are kept at a constant distance, the elastic curve will revert to the original sine-curve with one half-wave; this reversion to the stable type is analogous to the fall of the ball from the edge to the table. The set of neutral equilibria at the higher astatic loads may be represented in the analogy by a set of horizontal edges, one above the other.

Concerning the matter mentioned by Mr. Godfrey, of the stability of a thin circular cylinder or ring subjected to outside pressure, reference is made

to Part *C* in Mr. Halmos' discussion and to the papers listed in Section *G* in the Bibliography, particularly to Goupil's analysis. In this connection, an article by Hencky,* which appeared recently, may be mentioned; it deals with the stability of a circular ring against warping.

Concerning the coefficients of moments in slabs mentioned by Mr. Godfrey, reference is made to a recent paper on "Moments and Stresses in Slabs", by W. A. Slater, M. Am. Soc. C. E., and the writer.† The coefficient, 0.077, refers not to a slab, but to a double system of crossing-beams, as was discussed at the end of Article 31 (Formulas (158) and (159)).

The writer agrees with Mr. Godfrey concerning the importance of knowing the factor of safety and understanding its significance under the various conditions. As emphasized previously, the ultimate load, and, through it, the factor of safety, are dependent, in the case of medium slender and heavy structures, on the conditions after the proportional limit of the material has been exceeded. Further investigations, following the course indicated by Kármán and Southwell, are needed in this field.

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* "Kippsicherheit und Achterbildung an geschlossenen Kreisringen", *Zeitschrift für angewandte Mathematik und Mechanik,* v. 1 (1921), pp. 451–455.

† *Proceedings,* Am. Concrete Inst. (1921), pp. 431 and 438.

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WINTER OVERFLOW FROM ICE GORGING ON SHALLOW STREAMS

By J. C. Stevens,* M. Am. Soc. C. E.

With Discussion by H. B. Muckleston, M. Am. Soc. C. E.

Synopsis

Little has been written on the subject of ice gorging since the report of the St. Lawrence River Commission and the construction of the ice barrier to protect Montreal, Que., Canada. The overflow of certain Western streams, caused by ice gorging during the winter months, presents quite different phases of this question.

The occurrence discussed in this paper is not the overflow caused by ice jams brought about, during the spring break-up, chiefly by blocks of crystalline surface ice being arrested at some constricted part of the river channel and thus forming a temporary dam.

The ice gorge herein discussed occurs during the coldest part of the winter on streams too turbulent to permit the formation of crystalline surface ice; however, frazil and anchor ice are formed in such quantities that the stream becomes a viscous mixture. This results in a rise of the water surface and if the banks of the stream are too low to accommodate this increase of stage, overflow is inevitable.

Increasing the flow of such streams during the winter months by the release of artificially stored water presents an entirely new phase of the problem and merits careful analysis, especially where the overflow may become a menace to life and property.

The facts gathered by the writer during an investigation of the winter overflow from Madison River, Montana, and the effect of storage reservoirs

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thereon, are also true of other streams with similar characteristics and uses and should be of more than passing interest in the field of hydraulic engineering.

Statement of the Problem

Madison River rises in Yellowstone National Park and flows in a general northerly direction, uniting with the Jefferson and Gallatin Rivers near Three Forks, Mont., to form the Missouri. It flows through two agricultural valleys locally known as the Upper and Lower Madison Valleys. In these valleys the river banks are low, and near the lower end of each valley the river divides and subdivides into a network of many brush-lined channels.

In these many channeled parts of each valley, during the cold winter months, ice gorges of varying characteristics are formed. These gorges frequently cause the river to leave its channel entirely and flow across the valley floor, occasionally driving the residents from their homes and leaving the valley covered with solidified frazil ice many feet in thickness.

In 1913, the Montana Reservoir and Irrigation Company completed the Hebgen Reservoir. During the summer, this reservoir is operated for the purpose of supplementing the low-water flow of the Madison and Missouri Rivers for the benefit of the Prickly Pear Irrigation Project near Helena, and during the fall and winter for eight hydro-electric plants of the Montana Power Company located on those rivers.

The winter of 1916-17 was one of exceptionally sustained, moderately low temperatures, during which an unusual quantity of frazil and anchor ice was formed. This resulted in ice gorges and extensive overflow of the agricultural lands in both valleys. The question arose as to whether the ice gorging and the overflow were augmented by the operation of the Hebgen Reservoir and of the Madison Reservoir and the power plants in the canyon between the upper and lower valleys (Fig. 1). Accordingly, there was instituted a thorough study of the ice gorging and winter overflow phenomena of Madison River, both from a physical and an historical standpoint, as well as the effect of the operation of the reservoirs and power plants thereon. This was done in order to ascertain whether such operation was responsible wholly or in part for the overflow and to outline, if possible, some remedial measures.

Phenomena of Ice Formation

The temperature of a mixture of ice and water is always at 32° Fahr., as far as practical temperature measurements are concerned. It is a fact that small differences of temperature do exist; the range, however, is a matter of a few thousandths of a degree above or below the freezing point.

This condition is due to the latent heat of fusion of water. Whenever 1 lb. of water passes from the liquid to the solid state about 144 B. t. u. are liberated. Conversely, when 1 lb. of water in the form of ice passes to the liquid state, the same quantity of heat is absorbed in causing the molecular change from a solid to a liquid state and does not appear as a temperature

change in the body. Therefore, a mixture of ice and water maintains a balance of temperature; if it is subjected to a cooling influence, more ice is formed, which liberates just sufficient heat to keep the mixture at a constant temperature, and *vice versa*. It is of interest to note that from every acre of surface ice formed, 1 ft. in thickness, the same quantity of heat is liberated as in the burning of 18 tons of ordinary coal.

FIG. 1.

There is always a transfer of heat from a warm to a colder body. This transfer takes place by three processes, namely, radiation, convection, and conduction, all of which may be operating at the same time. The water in a river is cooled largely by convection. As the temperature of the air falls below that of the water, a transfer of heat immediately takes place from the warmer water to the colder air. The water at the surface is cooled by contact with the air and becomes denser, therefore, this layer of water sinks and is replaced by warmer water, thus setting up convection currents which gradually cool the whole mass.

When the body of water reaches a temperature of 39° Fahr., a reverse process sets in. At this temperature, water has its maximum density and expands as it is further cooled. Therefore, between 39° and 32° Fahr., the colder water remains at the surface in contact with the cold atmosphere.

Surface Ice.—As soon as the surface layer of water has reached a temperature of 32° Fahr., ice begins to form. From the water's edge the ice crystals begin to spread out over the surface, gradually extending toward the center of the stream. A nucleus of some kind is necessary for the formation of an ice crystal. Such nuclei may be tiny globules of dissolved air from the spray of tiny waves that break on the shore, snowflakes, particles of silt or sand, or any microscopic foreign matter in the water.

After a surface covering of ice is formed, the cooling of the water below it by convection virtually ceases, because the temperature gradient between the under surface of the ice and the water is indeed very slight. Continued cooling by conduction through the ice, however, goes on, but at a much slower rate. Thus, the thickness of surface ice increases at a continually decreasing rate, because the quantity of heat conducted through the ice decreases as the ice increases in thickness. An accumulation of snow on the ice greatly diminishes the thermal conductivity of the surface covering and also greatly decreases the rate of ice growth formed from the under side. A substantial covering of ice usually prevents the growth of any other form of ice beneath it, that is, frazil and anchor ice cannot form, because further heat losses from the body of water and the river bed are practically stopped. The water beneath surface ice is a few thousandths of a degree above freezing temperature, except, perhaps, the film of water directly in contact with the under side of the ice sheet. In cold climates, all modern power plants utilize this principle by securing a substantial reservoir above the power plant, and where they are so arranged, no matter how cold the weather, troubles from frazil ice in turbine wheels are practicaly eliminated.

In this respect, the effect of a covering of surface ice on a lake or reservoir may be likened somewhat to the effect of the glass cover of a hothouse in keeping the temperature of the water beneath it slightly above the freezing point.

Frazil Ice.—This form of ice is frequently called slush ice, spicular ice, mush ice, and needle ice. It is also a surface ice, and appears whenever the water is too turbulent for the formation of solid surface ice. Great quantities of this ice may be formed during protracted cold weather. It floats and has the general appearance of snow in the water, but on examination is found to consist of small spicules or needle-like crystals which have a tendency to cohere in masses. The entire water area of the river may become impregnated with frazil ice, forming a mixture much more viscous than water, which causes a retardation of velocity with a consequent increase of stage. Favorable conditions for its formation are turbulent water with cold, strong, up-stream winds and cloudy weather.

Anchor Ice.—Except at its place of formation, which is on the bed of clear shallow streams, anchor ice can hardly be distinguished from frazil ice. The cause of its formation is the rapid radiation of earth heat from the river bed

into space. It forms most rapidly on dark-colored stones, during clear cold nights, and it will not form under bridges, in the shade of overhanging trees, during cloudy weather, and seldom or never under surface ice or in water which is heavily loaded with frazil ice or silt. It is purely a product of radiation and, in this respect, forms under the same conditions as hoar frost.

With a slight rise of temperature during clear, sunny days, this anchor ice will rise to the surface. In rising, it often lifts and carries away the rocks and gravel to which it originally adhered. In appearance, anchor ice is granular and somewhat resembles honeycomb. Once formed, it may grow rapidly by an accumulation of frazil ice from the flowing water adhering to it, and when floating in the river, it cannot be distinguished from a mass of frazil ice.

Causes of Winter Overflow

It has been stated by local authority that the Madison is the only river in Montana which overflows from ice gorges during cold weather. This statement is not true. The Ruby, Boulder, and many small streams behave exactly in the same manner as the Madison in this respect. Floods on the Yellowstone, Missouri, and other streams are frequently caused by ice jams during the spring break-up. Ice gorges from frazil and anchor ice also form during the winter months, but the overflow caused by the latter is limited to the lower lands adjacent to the river.

The Madison is probably the largest river in the State in which winter overflow conditions are so pronounced. The reasons are not hard to find. Madison River has a fairly steep gradient throughout its course. In the two valleys under consideration, the banks are low, the river is shallow and wide, and the bed is strewn with boulders, cobblestones, and gravel. There are many low, brush-covered islands and bars, that form in some places a veritable network of channels, which condition is the natural result of the flow characteristics of this river, combined with the slope of and the gravelly soil composing the valleys. The flow of Madison River is comparatively uniform, and, as shown by Table 1, has only about one-fourth the range of the other streams

TABLE 1.

Stream.	Locality.	Maximum flow, in second-feet.	Minimum flow, in second-feet.	Percentage of maximum flow.
Yellowstone	Huntley	47 900	1 060	2.2
Jefferson	Silverstar	16 500	320	1.9
Gallatin	Logan	5 400	178	3.3
Madison	Plant No. 2	9 500	1 200	12.6

Other things being equal, a stream with extremes of flow will have a deeper channel in its self-made valleys than one with a low range of flow. Therefore, the Madison, with its low range of flow, has banks averaging about 3 ft. in height through the valleys.

The water surface of a stream carrying large quantities of frazil and anchor ice is raised by reason of the reduction in velocity caused by the increased

viscosity of the flowing mixture. A condition is soon reached in which the river changes from a turbulent to a placid stream. If the banks are high enough, this placid condition is reached before overflow occurs and surface ice is formed, which effectually prevents the further formation of frazil and anchor ice beneath it. Unfortunately, the banks of the Madison River are so low that overflow often occurs before this condition is reached and is the principal reason for the extensive winter overflow from ice gorges.

The immediate cause of overflow is the formation of ice gorges induced by low temperatures, and such gorges invariably occur during a protracted period of cold weather. When the river water is cooled to the freezing point, frazil ice immediately begins to run. If the nights are cold and clear, anchor ice will form, and if the days are clear, these masses rise and float away. If the days and nights are cloudy, anchor ice does not form, but a greater quantity of frazil ice is created. During storms, snowflakes and frozen spray falling into the water form nuclei that aid in the growth of frazil ice. Thus, whether the atmosphere is clear, cloudy, or stormy, if the air temperatures are lower than freezing, the river, once the water has been cooled to the freezing point, is continually manufacturing ice, the rate depending on the air temperatures below 32° Fahr.

In the Upper Madison Valley, the ice gorges usually begin just above the upper end of the Madison Reservoir. This part of the valley is broken into a network of channels, and there are a great many low brush-covered islands and bars and innumerable obstructions to flow. In the Lower Madison Valley similar conditions exist.

With the accumulation of frazil and floating anchor ice, these channels become completely choked and the river is virtually dammed with ice. Overflow is inevitable. First, the low sloughs and overflow channels are filled with water on which may form surface ice, and over which the river flows with its load of frazil. These channels soon are filled as high as the surrounding land, which causes the river to seek new outlets. In this manner the overflow spreads, and the water frequently leaves its natural bed altogether and flows over the surrounding bottom-lands. A water channel tends to choke itself with frazil ice; thus, the water is continually changing its path of travel and virtually raises the entire bottom to an irregular level of ice. While the cold continues, this frazil, exposed to the air, solidifies in huge masses, forming effective banks. There is practically no limit to the extent of overflow or ice accumulation that may occur as long as the critical degree of cold continues.

The surface ice at the edges is broken off by the rising water and floats down to add its volume to the gorge. These cakes frequently lodge edgewise and form an effective barrier for the accumulation of frazil and anchor ice. With a few days of rising temperature, the river begins to cut through and flows under the ice in its original channel, gradually clearing it of the accumulated ice. Oftentimes, banks of frazil 7 or 8 ft. high are left along the river's edges.

Two types of ice gorges were recognized on Madison River, namely, that designated as the "bridging gorge", in which little or no overflow occurs, and that called the "overflow gorge".

FIG. 2.—TYPICAL GORGE OF THE "BRIDGING" TYPE.

FIG. 3.—APPEARANCE OF RIVER AFTER ICE GORGE HAS BROKEN AND THE WATER HAS RETURNED TO ITS CHANNEL.

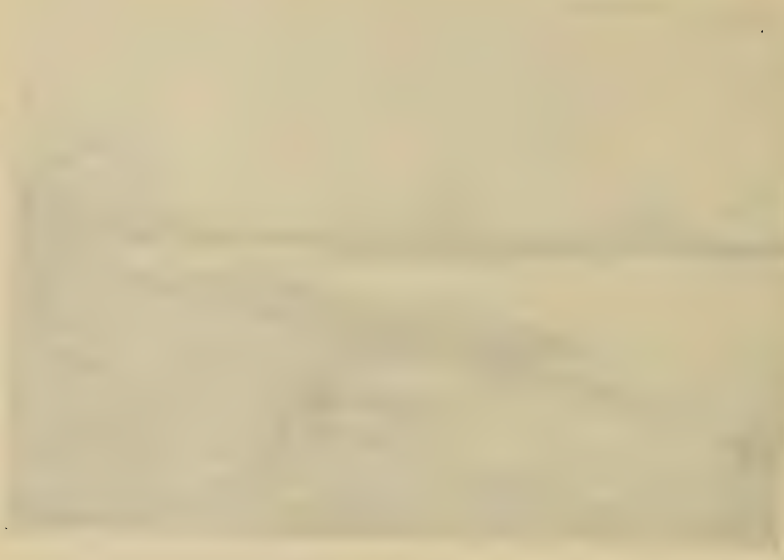
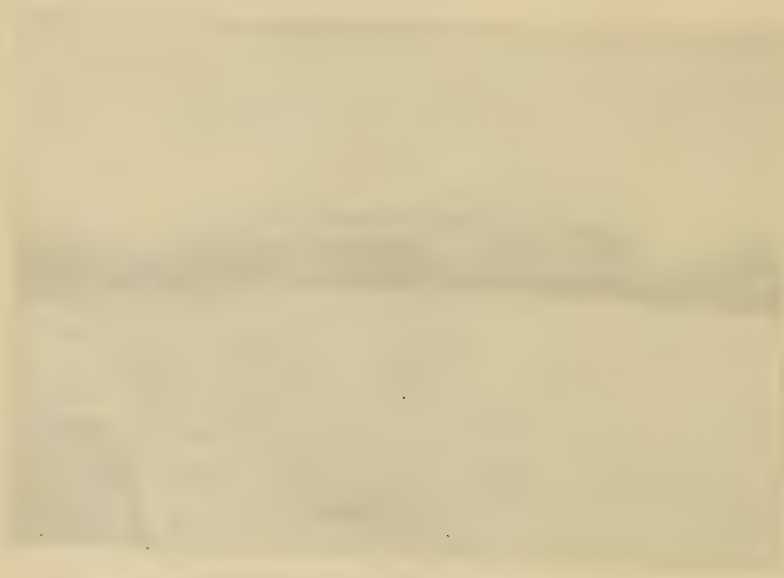


Fig. 4.—Frazil Ice Floating in River Channel.

Fig. 5.—Ice Gorge of the "Bridging" Type on Gallatin River.



The "bridging gorge" is caused by sudden and sustained extreme low temperatures, say, from −15° to −30° Fahr. This condition causes the maximum quantity of frazil ice to form, and the river is suddenly converted from a turbulent stream to a sluggish flowing mixture throughout its entire length. The river rises and is frozen over for long stretches of its length, thus preventing in these stretches the further formation of frazil and anchor ice. As long as the low temperature continues, the water flows beneath the ice bridge without overflowing its banks appreciably. Fig. 2 shows such an ice bridge with the water flowing beneath it.

The "overflow gorge" is caused by sustained moderate temperatures, say, between 15° and 25° Fahr. Such a condition leaves the river open practically throughout its entire length, causes considerable frazil and anchor ice, but not enough to form a "bridging gorge". Therefore, the formation of ice continues unabated, and the many channeled lower part of the valley becomes the gathering ground for all the frazil, surface, and anchor ice formed in the river above.

Between these two distinct types of ice gorges there is every possible intermediate type, the one merging into the other, causing varying degrees of overflow. The river is thus known to overflow one place one time, and some other place another time, in fact, every possibility of gorging and overflow between the two extremes is known to have occurred. So capricious are these overflows that, during the winter, stock is never allowed to pasture in the bottomlands. In the history of these valleys, thousands of head of cattle have been trapped and lost in the winter overflow.

By interviewing the old settlers of the valleys, it was found that ice gorges and overflows were to a greater or less extent of annual occurrence. Winters that stand out especially in this respect are those of 1867, 1875, 1883, 1898, 1910, and 1917.

In the Lower Madison Valley in the decade, 1890-1900, iceboats were successfully used by a colony of sport-loving Englishmen. The ice overtopped the fence posts, giving clear stretches 10 miles or more in extent. In fact, it appears that the limits of the 1916-17 overflow had at times been equalled if not exceeded.

Effect of Reservoir Operation

Apart from an increase in winter flow, the effect of a reservoir in such a stream is first to diminish the quantity of ice below it. For example, consider a turbulent river, 100 miles in length, with a valley at the lower end. In its natural condition the river pours into this valley all the ice formed throughout its length.

Now, if a substantial reservoir is built 25 miles above the valley, it will constitute an ice barrier for the 75 miles of river above it, since no frazil ice can pass the reservoir. Moreover, the water from this reservoir is slightly above freezing temperature and frazil ice cannot form until the water has been cooled to freezing in the channel below. It is obvious, therefore, that after the construction of the reservoir the quantity of ice reaching the valley

will be limited to that formed in the lower 15 or 20 miles of river, whereas, prior thereto, 100 miles of river contributed its ice to the valley.

The first effect of increasing the flow is to diminish the quantity of anchor ice that may form, due to the increased depth of water reducing the quantity of heat that may be radiated directly from the bed of the stream.

The second effect is to diminish the turbulency and encourage the tendency of the water, at freezing temperature, to remain on the surface. Thus, the formation of frazil ice is more likely to be confined to the surface of the moving water, which is not the case when the water is brought to a uniform freezing temperature by thorough mixing.

The third effect is to increase the quantity of frazil ice about in proportion to the increase in the area of the water surface.

The fourth effect is to increase greatly the transporting power of the current, thus enabling the river better to carry its icy load without gorging and, therefore, delay the formation of the gorge until a rise of temperature destroys the tendency of the ice to gorge. The increased flow tends to cut out the gorge quicker with a slight rise in temperature; in other words, the river becomes more sensitive to the temperature changes and the duration of the overflow is less than with the normal flow. For example, consider a typical cross-section of Madison River, as given in Table 2.

TABLE 2.

Gauge, in feet.	Mean depth, in feet.	Width, in feet.	Discharge, in second-feet.	Area, in square feet.	Mean velocity, in feet per second.	Transporting power.
2.0	1.84	177	880	325	2.60	1
2.5	2.19	186	1 470	425	3.45	6
3.0	2.80	195	2 350	545	4.31	21
			Increase in Percentage:			
2.0 to 2.5	19	5	67	31	33	500
2.0 to 3.0	52	10	167	68	66	2 000

It is stated in textbooks on hydraulics that the weight of bodies which can be moved by a current varies as the sixth power of the velocity, hence the increase in transporting power, corresponding to an increase of 67% in discharge, is 500%, and an increase of 167% in the discharge increased the transporting power 2 000 per cent. Of course, when the river becomes loaded with frazil ice, the velocity is somewhat reduced and the river rises, the flow remaining constant. With the increased flow and a loaded river, the full effect of the increase in transporting power is not felt, but, nevertheless, there is a decided increase in its transporting power and its ability to prevent and to cut out gorges. If the quantity formed varies as the surface exposed, there is only a 10% increase of frazil ice with 167% increase of discharge.

The change in the winter régime of a river by reason of the operation of reservoirs, results from five causes. The effect of four of these causes is to diminish the quantity of ice formed, and only one has the effect of increasing it; hence, the net effect of the construction and operation of reservoirs on the

Madison River was to diminish the quantity of ice and ice gorging. As applied to Madison River, this fact was abundantly verified by the testimony of old settlers still living in the valleys.

The next point of inquiry is whether a reduction in the quantity of ice might not increase the overflow in some manner by making the general type of gorge conform more nearly to that of the "overflow gorge" in distinction to the "bridging gorge", as previously defined. This question is intimately associated with temperatures and windy, clear, or cloudy days and nights. For any given set of climatic conditions, it is acknowledged that the effect of reservoir operation on the Madison River has been to reduce the total quantity of ice, to delay the formation of gorges, and to hasten the return to normal conditions. This effect results from physical causes which, if valid for one part of the valley, must be valid for all parts. What then produced the abnormal overflow in the Madison Valleys in 1916-17? The answer is readily found in abnormal climatic conditions.

The deduced mean natural flow of the Madison River at Plant No. 2, situated between the Upper and Lower Valleys (Fig. 1), during the months, November to March, each winter, and the increases in flow resulting from the operation of the Hebgen Reservoir, are given in Table 3.

TABLE 3.

Winter season, November-March.	Deduced mean natural flow, in second-feet.	Additional flow from Hebgen Reservoir, in feet.	Percentage of increase.
1914-15	1 600	440	28
1915-16	1 540	520	34
1916-17	1 440	940	65
1917-18	1 420	610	43
1918-19	1 460	1 090	75
1919-20	1 170	180	15
1920-21	1 310	110	8

The Hebgen Dam is about 60 miles below the source of the river and by creating a reservoir about 20 miles in length, has the effect of eliminating approximately 100 miles of river which before the construction of the dam contributed large quantities of ice to the lower valleys. Ice gorges now form above the reservoir as in previous years, but this mobile ice is melted in its passage under the surface ice of Hebgen Reservoir. The water issuing from the dam is free from ice and its temperature is slightly above the freezing point; therefore, no ice can form in this river until the water has again been cooled to the freezing point. It has been found by observation that no appreciable quantity of frazil ice, even in extreme temperatures, begins to form within 10 miles below the dam.

The effect of the Hebgen Reservoir, therefore, aside from the question of increased flow, has been to diminish the quantity of ice reaching the Upper Madison Valley. For the same reasons, the Madison Reservoir which is at the head of the canyon between the two valleys (Fig. 1), has the effect of still further diminishing the quantity of ice in the Lower Valley by eliminating

about 80 miles additional of ice-producing river channel. Hence, before the construction of either reservoir, the Lower Valley was the gathering ground for the ice produced in 150 miles of river. By the construction of the reservoir, the length of ice-producing channel has been reduced to about 25 miles.

During the winter of 1916-17, as a result of the operation of the Hebgen Reservoir, the mean increase in flow was 65%, and, during certain periods, it was more than doubled. Yet the increased quantity, if no ice was present, is only about one-fourth the capacity of the river channel before overflow occurs.

Ice-Forming Factor

The real cause of the unusual overflow of 1916-17 in the Madison Valley was abnormally sustained moderately low temperatures. The rapidity with which ice will form is dependent far more on the drop in temperature below freezing than any other factor, provided that drop is not sufficient to form a "bridging gorge" throughout the length of the river and thus prevent the further formation of ice. Clear days and nights, cloudy weather, winds, and snows, all have their effect, but they are of minor importance in comparison with the temperature gradient.

The quantity of ice formed will depend on the amount of heat lost from the water. The heat lost after water has been cooled to 32° Fahr. and ice has begun to form is the latent heat of fusion. Virtually, 1 lb. of water is frozen for every 144 B. t. u. extracted from the water. Practically, the same amount of heat is lost in 1 day with a drop in temperature of 10° as is lost in 2 days with a drop of 5°; hence, the number of degree-days below freezing in any period will be the measure of the quantity of ice formed during that period.

When the air temperature is above the freezing point, heat passes from the air to the water, and latent heat is absorbed by the water with a reduction in the proportion of ice. The quantity of ice thus lost is also measurable by the degree-days above freezing. Hence, in any period, the net quantity of ice is measured by the excess of degree-days below freezing over those above that temperature. These relations, of course, only hold as long as the river remains open and a mixture of ice and water may be considered.

Fortunately, long-time records of temperature were available at Bozeman (and Fort Ellis), Mont. These records beginning in 1868 were practically continuous to date. A comparison of temperature records at Bozeman with those obtained in recent years at Madison Plant No. 2, at Ennis, and at Three Forks, shows that the Bozeman records represent very accurately the temperatures in both the Upper and Lower Madison Valleys, and may be safely used to fix relative yearly variations.

The mean daily temperatures for the months of November to March, inclusive, for each winter, were plotted. The winter season was considered to begin on the first date in the fall that the mean daily temperature fell below freezing and to end with the date it rose above freezing in the spring. The areas of these temperature curves below freezing in excess of those above freezing were found by means of a planimeter. These areas gave for each winter season the number of degree-days below freezing in excess of those

above freezing. This quantity was called the ice-forming factor for that season. Figs. 6 and 7 show the temperature curves for the winters of 1871-72 and 1916-17, respectively. Fig. 8 shows the ice-forming factors for each of the 53 years of record, arranged in chronological order, and Fig. 9, the same factors arranged in order of magnitude. With the exception of the winter of 1871-72, it is seen that the winter of 1916-17 shows the greatest ice-forming factor during the entire 53 years.

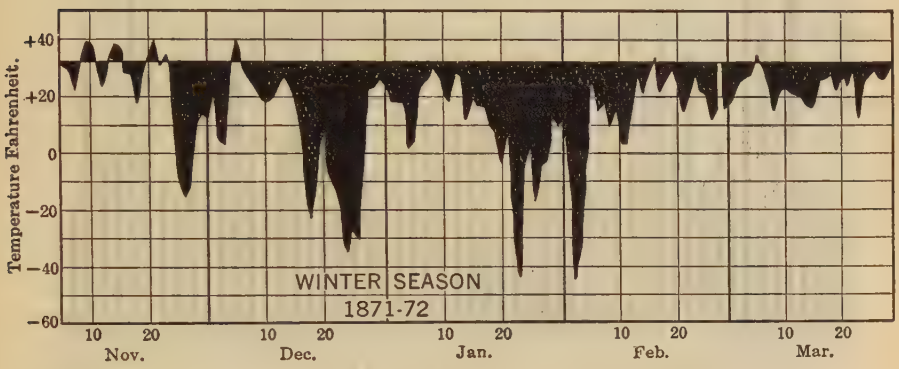


FIG. 6.

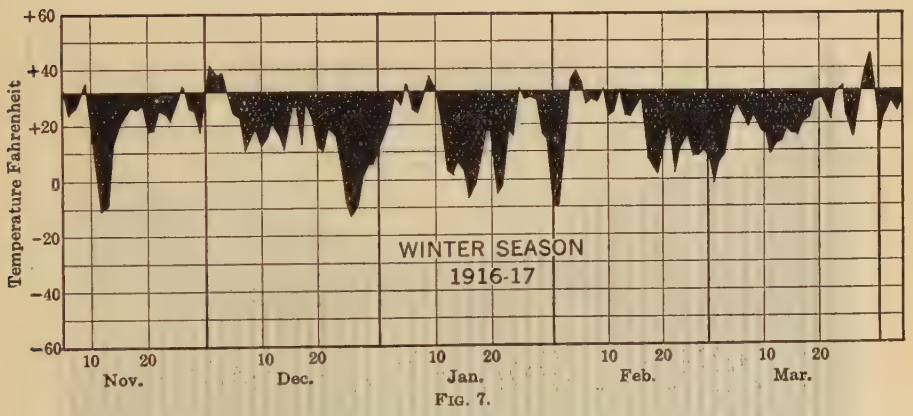


FIG. 7.

The ice-forming factor alone is no measure of the overflow, but merely indicates the net quantity of ice-forming influence from temperatures that obtained during the winter. Overflow results from the type of gorge produced, and one must look to the daily temperature curves themselves in order to judge of the nature of the gorges that would be formed. A comparison between Figs. 6 and 7 shows at a glance why the gorge produced during the winter of 1916-17 was of the overflow type, while that of 1871-72 was more likely to have been a "bridging gorge" which did not produce any unusual overflow.

From this analysis the conclusion was inevitable that the year showing the greatest ice-forming factor and, at the same time, in which the highest temperatures obtained, was the one producing maximum overflow. Of all the years of record, the winter of 1916-17 was the most abnormal in this regard.

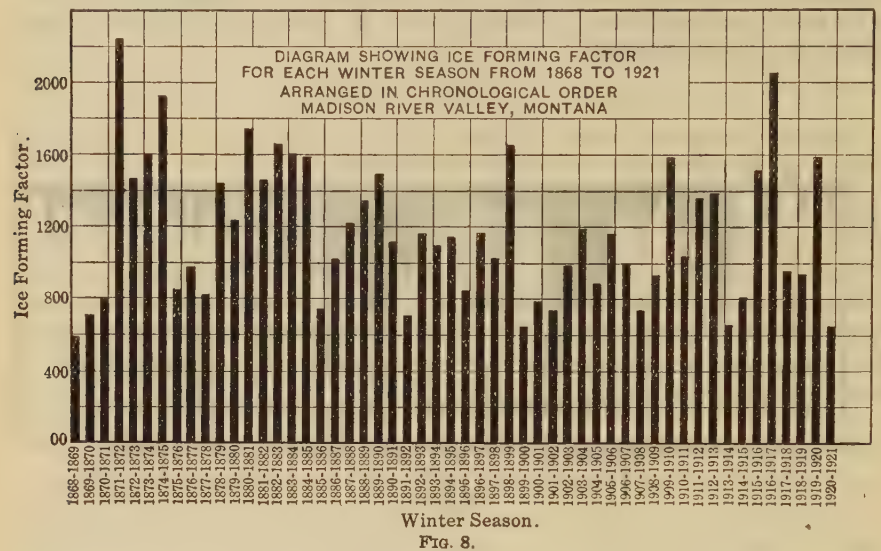


FIG. 8.

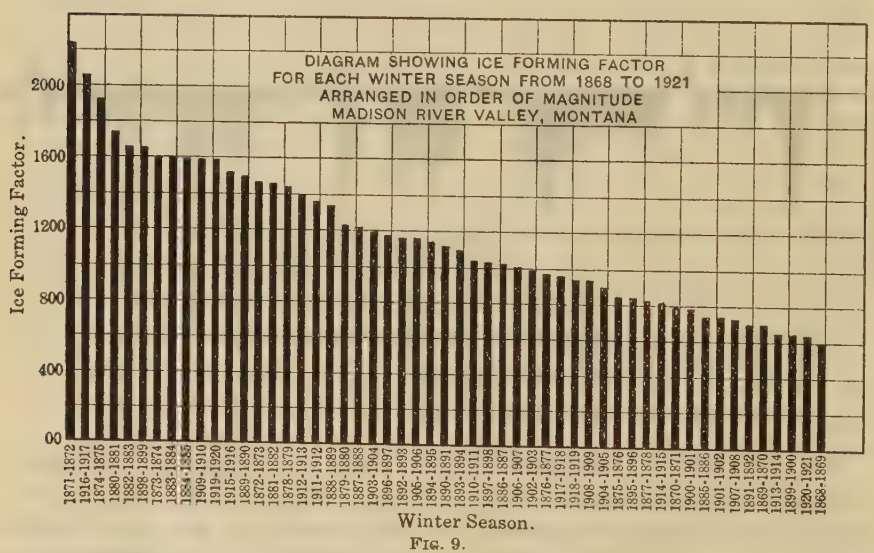


FIG. 9.

FLUCTUATIONS CAUSED BY ICE GORGING

Obviously, when an ice gorge has formed and overflow has commenced, great fluctuations in flow and in river stage are produced. During the formation of the gorge, a great quantity of water is converted into ice and remains in

FIG. 10.—FRAZIL ICE LEFT ON AGRICULTURAL LANDS AFTER RIVER HAS RETURNED TO ITS CHANNEL.

FIG. 11.—BROKEN ICE LINE ON BANK SHOWS HEIGHT TO WHICH WATER WAS RAISED AT TIME OF FORMING OF GORGE.

temporary storage until released by a rise in temperature. In fact, it was the inability to secure water for power-plant operation in the canyon between the two valleys, which led to the construction of the Madison Reservoir.

In order to secure definite data regarding the fluctuations caused by gorging, two automatic water-level recorders were used, one at the Varney Bridge in the Upper Valley and the other at the Three Forks Bridge in the Lower Valley. To insure records during the freezing temperatures, the floats of each recorder were arranged to rest on oil. In this manner, water levels were secured at both places during periods of gorge formation.

At both places, the gorges formed were of the "bridging" type, with the water flowing under the ice and carrying varying quantities of frazil ice from the open river above. Once the gorge was formed, the fluctuations follow broadly the variations in temperature.

It is the general tendency for water to rise with a fall in temperature. This is explained by the slowing up of the current due to an increase in the quantity of frazil ice. A drop in temperature is also coincident with a reduction in the flow, due to the temporary increase of water stored in the form of ice in the river channels, and in the increased areas of flowing water.

One effect of the fluctuations, which is damaging to meadow lands, is the cutting and the removal of sod. Evidence of this was plainly seen near the head of the Madison Reservoir and in certain areas near the head of the gorges in the Lower Valley. In some of the meadows and pastures, the sod had been badly cut by the ice. Examined early in June, 1917, the peculiar phenomena were seen of irregular patches of sod from 1 sq. yd. to 50 sq. yd. in area resting on top of several feet of ice, the explanation of which is simple.

The first overflow covered the land with a sheet of water, which, in freezing, became attached to the sod. A second flooding covered this ice 1 ft. or more in depth. The ice on the bottom was cracked and broken in irregular patches and rose to the surface carrying with it the sod on which it was first formed. Later, as the gorge increased, the depth of water became greater and the current flowing under the surface ice, being impregnated with frazil ice, soon filled completely the space between the surface ice and the ground. The ice under the sod is protected and is the last to disappear when the ice melts in the spring. Hence, these patches resembled huge toadstools of ice with sod caps.

Some of these ice cakes with sod adhering to them float away and in melting deposit the sod in adjacent fields. Generally, however, by becoming a part of the surface ice in the ponded water immediately above the ice gorge, they are held in place and, in melting, deposit the sod near the place from which it came, except that it is usually moved a few inches down stream or tilted and broken.

There are evidences throughout both valleys that this process is a natural phenomena of overflow from ice gorges and has been going on from time immemorial.

Remedial Measures

It was previously pointed out that the banks of the Madison River are not sufficiently high to retain the water in its channel while the stage increases as

a result of retarded velocities due to its ice load. Overflow often occurs before the surface can bridge over with ice and thus prevent the further formation of frazil and anchor ice.

The obvious remedy is to increase the height of the banks by dikes. In the Lower Valley, as a result of this investigation, a diking district was organized by the land-owners affected and a dike 11 miles in length was built. The top of the dike was constructed 10 ft. above low water, and the dike was placed as near the river bank as possible consistent with protection from under-cutting. Where necessary, ordinary culverts without trap-gates were built through the dike for drainage. The effect of this will be to prevent future overflow, but it will be necessary to irrigate some of the lands which previously were watered from these overflows. Undoubtedly, the winter overflow was a benefit in the raising of wild hay crops, and whether or not all overflow should be prevented can only be determined on the merits of each tract or area as governed by its local conditions.

DISCUSSION

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—The phenomena exhibited in ice and ice formation are of great material interest to hydraulic engineers and especially where their work is in altitudes or latitudes sufficiently high to make trouble from ice formation a matter of annual occurrence.

In spite of the valuable research work already done, there is room for a great deal more, and contributions to the knowledge of the subject are a welcome addition to science.

The *modus operandi,* causes, and effects of ice formations are still obscure in many phases, notwithstanding its apparent simplicity. The formation of the so-called frazil ice is one subject on which the last word has not been said. It is possible to cool water to a temperature much below its freezing point, provided the water is absolutely still, for example, in a glass jar. Sudden agitation by shaking or even inverting the jar will cause it instantly to congeal. A similar action may be noted sometimes when large quantities of frazil ice float in practically still water without solidifying. Very slight agitation, such as that caused by the sudden opening of a submerged valve may result in the instantaneous freezing of the whole into a solid mass.

Anchor ice forms on the bed of shallow streams; but shallowness is a relative term, since anchor ice has been observed under 10 ft. of water. A similar formation sometimes occurs in flumes, particularly in those constructed of metal or concrete, and in metal pipes entirely exposed to the atmosphere. Where the pipes rest on earth, it has also been noticed that anchor ice forms only on the part exposed to the air. It will form under bridges if conditions are right.

The size of stones which at times are lifted by anchor ice, is astonishing; one weighing 52 lb. has come under the writer's observation. It might also be noted that these stones may get into all sorts of unexpected places; in one case, a pear-shaped boulder became lodged in the groove of a Stoney gate, and had to be broken before it could be removed.

The statement that the transporting power of water varies as the sixth power of the velocity needs qualification. First, it refers to bodies of higher specific gravity than water; second, it refers to movement of any kind, not merely floating or apparent floating, such as may occur with suitably shaped objects in violent streams; as, for example, the locomotive which serious witnesses were prepared to swear floated on the Johnstown flood. In so far as ice, or other light material is concerned, the transporting power depends on the depth. The eroding power of water in motion is another matter.

A phenomenon which is observed in rivers in the "chinook belt" east of the mountains, is known as over-flooding. The less turbulent streams usually freeze over more or less completely, early in the winter. As the result of a continued cold snap, the discharge is somewhat reduced and, as a consequence, the ice in mid-stream sags perhaps several inches or even feet. A chinook increases the discharge rapidly to a quantity which the channel under the ice cannot accommodate, and, as a result, the water flows in a thin sheet over the

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ice and freezes. This results in extremely thick ice, especially on the edges, which is prone to remain solid long after the ice in the center is rotten. When it finally does float away, it is in excellent shape to cause jams in the first suitable place. In one case, on a stream much used for skating, this phenomenon was a regular afternoon occurrence on warm days. Skaters going up the river would meet a thin sheet of water about an inch or less in depth flowing down. This water was always frozen the next morning; but if the air was warm enough, that is, just above freezing temperature, it would always recur about three or four o'clock in the afternoon.

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Paper No. 1492

THE AREA OF WATER SURFACE AS A CONTROLLING FACTOR IN THE CONDITION OF POLLUTED HARBOR WATERS

By Richard H. Gould,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. C. Purdy, Allen Hazen, Warren R. Borst, Kenneth Allen, H. W. Streeter, George A. Soper, H. W. Clark, Gordon M. Fair, and Richard H. Gould.

Synopsis

The sources of oxygen required to meet the demand of the polluting matter in the waters of New York Harbor are discussed in this paper. It is shown that the air is the most important source of this oxygen which is obtained by direct absorption through the water surface. The conclusion is, that under aerobic conditions, oxygen dissolved in the harbor waters will be depleted by the demand of the polluting matter until a condition is reached where oxygen is absorbed from the air at the same rate at which it is abstracted by the fermenting material in the waters.

During 1920, the writer had occasion to make a number of tests and observations on the pollution of the waters of parts of New York Harbor and some of its tributaries. As a result of this work, and of other studies, he has been impressed by the tremendous part apparently played by atmospheric oxygen in supplying the oxygen demand of the polluted harbor waters and in preventing obnoxious conditions. The importance of atmospheric oxygen in this connection has been stressed by other writers, notably by Dr. W. E. Adeney in his report to the New York Metropolitan Sewerage Commission, published in 1912. Others, however, have credited this factor with little influence on harbor conditions. In advancing opinions in support of the former view, the writer recognizes that the basic data available for study are meager, and this paper is presented with the purpose of developing additional information.

* Prin. Asst. Engr. with James H. Fuertes, New York City.

The data under discussion were secured by the writer in a series of dissolved oxygen analyses, covering a period from July to December, 1920, made in connection with a report by James H. Fuertes, M. Am. Soc. C. E., to the municipal

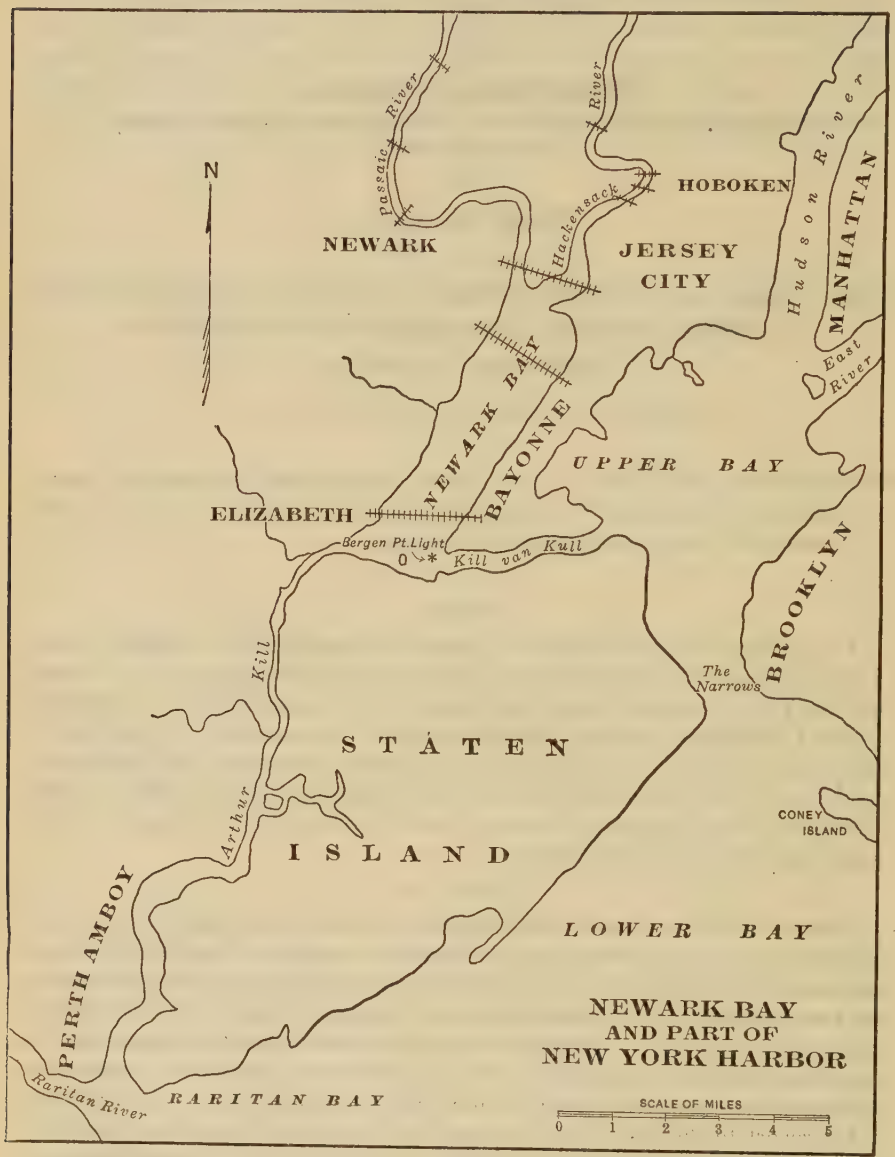


FIG. 1.

authorities of Elizabeth, N. J. These data were taken principally on the Jersey side of New York Harbor, mostly in Newark Bay, Kill van Kull, Arthur Kill, Raritan Bay and tributary streams (Fig. 1). The analyses were carried out

in accordance with methods used by the Metropolitan Sewerage Commission, and the large amount of data given in the valuable reports of this Commission have been used freely to supplement the new data which necessarily were limited.

In the course of the investigations, it was observed that, at times, the dissolved oxygen content of the water changed rapidly from day to day and that the changes seemed to be due primarily to the wind, that is, following a brisk wind which caused considerable wave disturbance, the dissolved oxygen content of the waters might be much higher than is usual in calm weather under similar conditions of temperature and dilution with sea water. For example, on August 24th, 1920, the dissolved oxygen in Newark Bay, at the Lehigh Valley Railroad Bridge, was 14% of saturation, and eight days later, at this point, it was found to be 75% of saturation. In the first case, there was little wind and wave disturbance, whereas in the latter there was much wave motion caused by a brisk northwest wind. At another time, in the Arthur Kill, at Elizabeth, N. J., the dissolved oxygen value increased from 19% at 9:25 A. M., to 52% at 4:00 P. M. of the same day. No doubt, these are extreme cases, but they indicate the importance of the wind and possibly other natural phenomena on the dissolved oxygen content of the harbor waters.

The fluctuation of dissolved oxygen values, in the Arthur Kill, at Elizabeth, is shown graphically on Fig. 2 for certain days in the period from July to December, 1920. The percentage of sea water present and the temperature of the water are also shown on the diagram. Average daily wind velocities, as obtained by the Weather Bureau on the roof of the Whitehall Building in Lower Manhattan, are also shown. This station is about 6 miles from Newark Bay and at considerable elevation, hence the velocities obtained do not represent actual wind velocities at the surface of Newark Bay. They do show, however, the relative fluctuation in the wind currents, and are valuable, at least, to that extent.

It may be noted that the majority of high saturation values follow or are concurrent with relatively high wind velocities, and, conversely, that the lower saturation values follow lower wind velocities. A quantitative relation between wind velocities and dissolved oxygen content is not apparent from present data, but such a relation could be obtained, no doubt, if sufficient data were available on wind velocities and their direction and on dissolved oxygen content. Although this information would be interesting and of some value, engineers are concerned to a greater extent in the quiescent condition of the harbor waters. The critical conditions for fish life and for the production of nuisances occur when the dissolved oxygen is depleted to the greatest extent.

The chief interest in the effect of wind velocity on the dissolved oxygen content, as far as this paper is concerned, is the attention it draws to the importance of atmospheric oxygen in the maintenance of dissolved oxygen values in the waters. It seems probable that the chief effect of the wind is to ruffle the water surface and, therefore, increase the area exposed to the air. That such decided changes in the dissolved oxygen content occur when the area of the water surface is thus increased, leads to the opinion that oxygen absorbed from the air even through the quiescent water surface may be of considerable

importance. An effort was made, therefore, to define the relative importance of the sources of oxygen to the polluted waters by a study of conditions in Newark Bay.

Newark Bay is a shallow basin, with an average depth of about 6.9 ft. at low tide and a water surface area of about 8.06 sq. miles. Its location in reference to New York Harbor is shown on Fig. 1. The Hackensack and Passaic Rivers flow into Newark Bay at its upper or northern end, and their outlet to the ocean is through two channels, namely, the Kill van Kull, connecting Newark Bay with Upper New York Bay, and the long reach of the Arthur Kill, connecting Newark Bay with Raritan and Lower New York Bays. The waters in the tidal prisms of Newark Bay and of the Passaic and Hackensack Rivers flow back and forth through the two "Kills". According to the U. S. Coast and Geodetic Survey, 84% of these waters pass through Kill van Kull and 16% through Arthur Kill.

The waters of Newark Bay are heavily polluted. The inflowing Passaic River has been in a septic condition for years, as it receives the untreated sewage from Paterson, Passaic, Newark, and smaller communities. The Hackensack River is polluted to a lesser degree; however, it carries considerable waste material, and all the bordering cities, Jersey City, Bayonne, Elizabeth, and Newark, as well as the Borough of Richmond, New York City, discharge a part of their untreated sewage into the waters of Newark Bay. The waters of Upper New York Bay are polluted by sewers draining into New York Harbor, and tidal currents force a part of these waters into and out of Newark Bay.

An estimate was made of the quantity of this waste material entering the Bay and also its probable oxygen demand for the period of its retention in the Bay waters. The visible sources of oxygen to meet this demand were then measured and estimated to determine the relative importance of the various oxygen sources in preventing obnoxious conditions in the Bay.

The volume of the polluting material can be estimated approximately from the population whose wastes flow into the waters. For purposes of the computation, all the sewage flowing into the tidal waters of the Passaic and Hackensack Rivers and Newark Bay was included. The City of Elizabeth lies at the junction of Newark Bay and Arthur Kill, its sewage being carried north into Newark Bay on the flood tides and south into Arthur Kill on the ebb tides. For present purposes, one-half the volume of its sewage and that from a joint municipal sewer discharging at Elizabeth are considered as being tributary to Newark Bay.

The determination of the oxygen demand of the sewage which finds its way into the Bay is more complicated. This demand is dependent on the strength of the sewage, on the length of time it is in contact with the waters of the Bay, on the temperature, and on the nature of the bacterial actions taking place.

In a report to the Metropolitan Sewerage Commission published in 1912, Adeney estimates that the total oxygen requirement of settled New York sewage, at 0° cent., and 760 mm. barometer, would be not more than 250 cu. cm. per liter, and that about 60% of this amount would probably represent

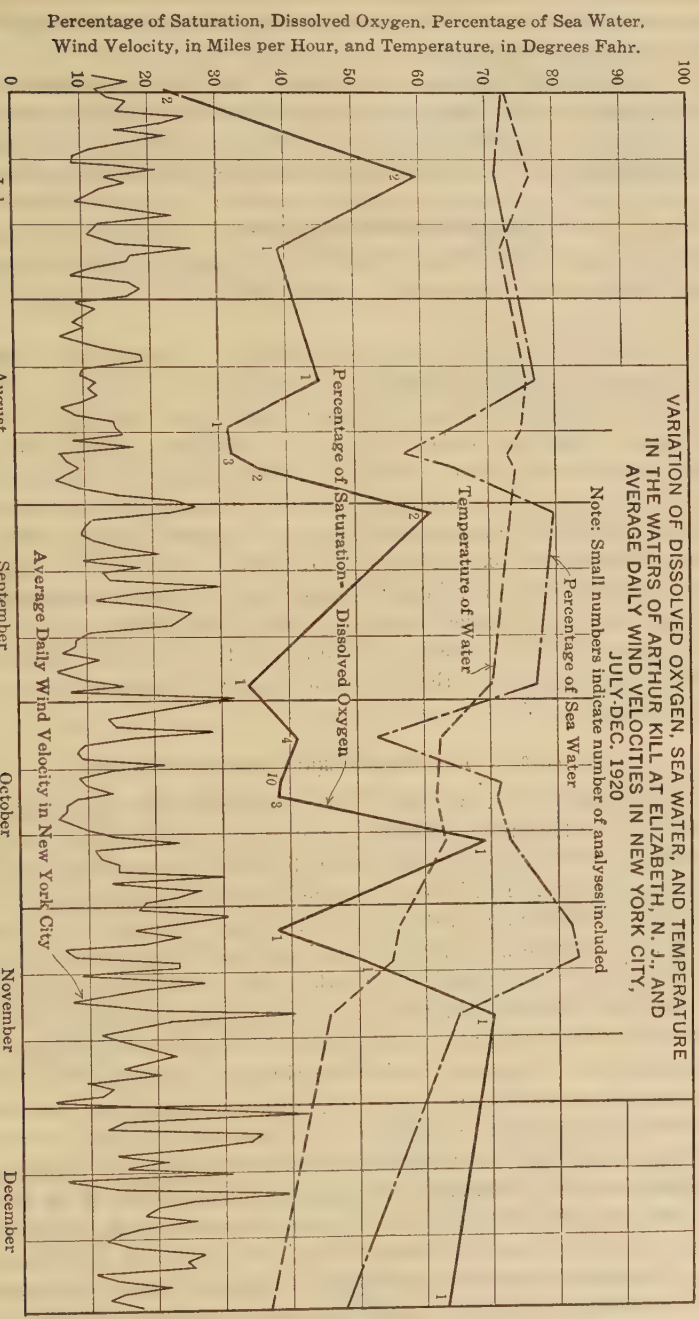

Percentage of Saturation, Dissolved Oxygen, Percentage of Sea Water, Wind Velocity, in Miles per Hour, and Temperature, in Degrees Fahr.
VARIATION OF DISSOLVED OXYGEN, SEA WATER, AND TEMPERATURE IN THE WATERS OF ARTHUR KILL AT ELIZABETH, N. J., AND AVERAGE DAILY WIND VELOCITIES IN NEW YORK CITY, JULY-DEC. 1920
Note: Small numbers indicate number of analyses included
Percentage of Sea Water
Temperature of Water
Percentage of Saturation—Dissolved Oxygen
Average Daily Wind Velocity in New York City
0
10
20
30
40
50
60
70
80
90
100
July
August
September
October
November
December

Fig. 2.

the demand during 48 hours under actual harbor conditions in summer, with a temperature of 25° cent. This latter figure corresponds to about 570 cu. cm. of oxygen per gallon of sewage.

The Metropolitan Sewerage Commission has made a number of small-scale experiments on the oxygen demand of New York sewage, with different dilutions of sewage and fresh water, the results of which varied considerably with the dilution used. From its work, however, the Commission estimated that about 1 428 lb. of oxygen would be required to oxidize 1 000 000 gal. of raw sewage. This is about 80% of that estimated by Adeney for the demand of settled sewage in New York Harbor. The estimate of the Metropolitan Sewerage Commission corresponds to about 455 cu. cm. of oxygen per gallon of sewage.

It is well to bear in mind the great difficulty in reproducing harbor conditions in small incubation samples. The dilution of sewage with harbor waters is uncertain, and this is also true of the actual time during which the sewage stays within the harbor limits. An additional consideration is the probable existence of a rather strong bacterial culture in the waters of the harbor. It is quite usual to see a marked brownish floc in the waters of Newark Bay, similar in appearance to that of activated sludge. More reliable data are needed on the oxygen demand of sewage in harbor waters, but with the limited information available it seems that Adeney's estimates for the oxygen requirement of settled sewage in New York Harbor may be accepted for the present. In the following computations, therefore, the oxygen demand for sewage entering Newark Bay has been taken at 570 cu. cm. of oxygen per gallon of sewage.

Absorption of Atmospheric Oxygen by Water

The differences of opinion regarding the quantity of oxygen absorbed by water have hinged largely on whether or not oxygen dissolved by the top layers could stream or be diffused or otherwise distributed throughout the body of water at lower depths. In his report to the Board of Estimate and Apportionment of New York City in 1911, Earle B. Phelps, Affiliate, Am. Soc. C. E., showed that if Ficks' law of diffusion was applicable, oxygen would not penetrate to any great depth, and that little oxygen could be absorbed by the water. He also estimated, in the same report, that the upper 12 ft. of the harbor waters were thoroughly mixed every 1.1 hours by the influence of tides, wind, shipping, etc. As a result of his experiments, Adeney held that dissolved oxygen would stream from the top to the bottom of the liquid, maintaining a nearly uniform percentage of saturation throughout the depth of the water.

The latter view does not seem to be inconsistent with the results obtained by the Metropolitan Sewerage Commission, which indicated, as a rule, no marked and general difference in the oxygen values near the surface and those toward the bottom. In Newark Bay, there seems to be little question but that dissolved oxygen is uniformly and rapidly mixed throughout the depth of the water. The tidal water coming in through the deep channels of the Kills is spread out like a fan over the shallow flats of the Bay. It is further mixed by the close-standing piles of the railroad trestles crossing

the Bay, and by the uneven bottom caused by ship channels and obstructions.

The quantity of oxygen capable of being absorbed will depend on the temperature, salinity, and percentage of saturation of the liquid. Water devoid of dissolved oxygen when exposed to the air will absorb oxygen rapidly at first, the rate of absorption decreasing gradually to a low value when the capacity of the water has been reached. A number of analyses made by the Metropolitan Sewerage Commission, Mr. Rideal, and the writer, have been recomputed,

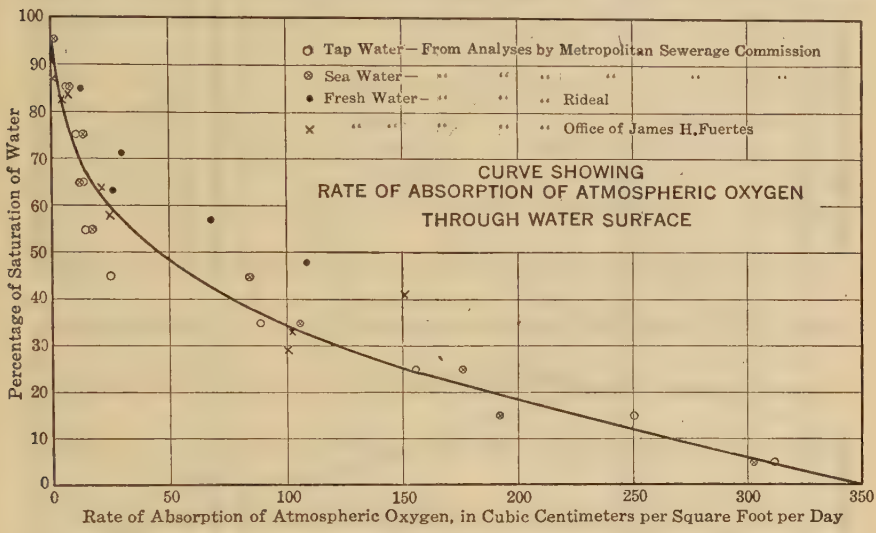


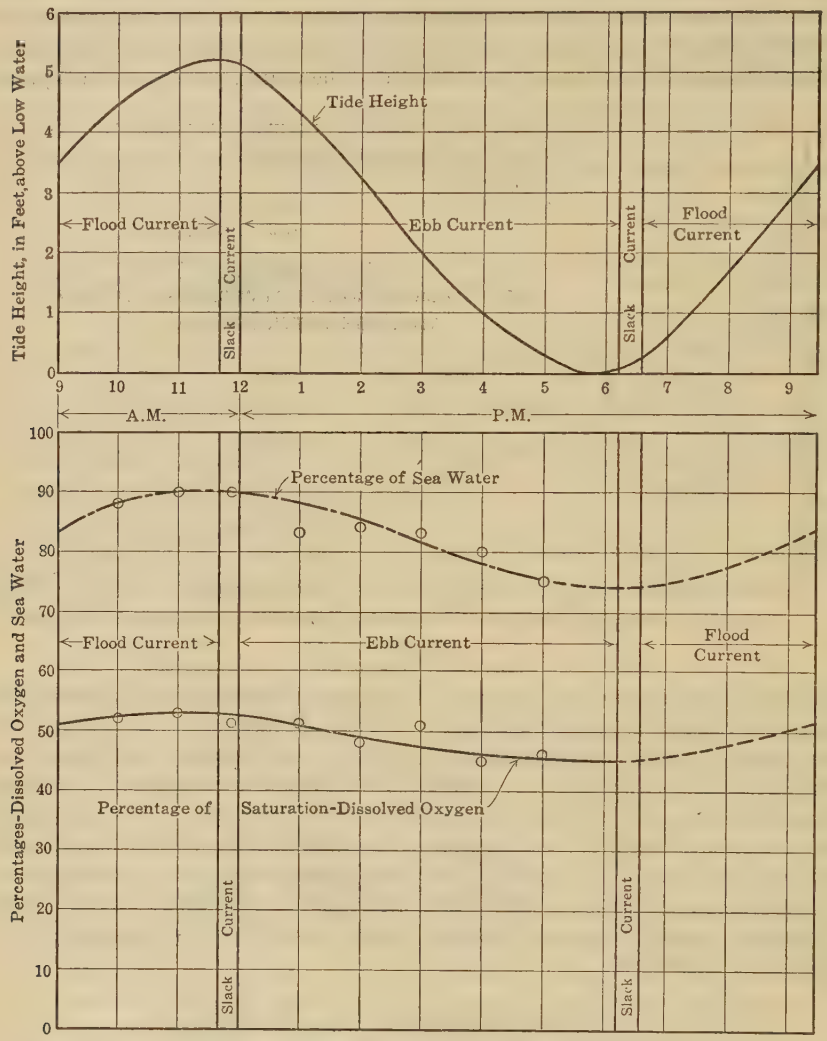
FIG. 3.

and the results are plotted in Fig. 3. The rate of absorption of atmospheric oxygen is shown for different saturation values of the water. No attempt has been made to apply a temperature correction as the data were not sufficiently extensive to show that this is necessary. The temperatures used in the analyses were about the same as those of the harbor waters in summer. Values are given for salt water as well as fresh water, no appreciable difference being apparent between the rates of absorption of the two waters when each is estimated on its saturation percentage. The values given being obtained from small-scale laboratory experiments are for quiet waters unaffected by wind and other disturbing influences.

Oxygen Supplied by Tidal Waters

One of the interesting and rather surprising facts disclosed by investigation was that the tidal waters flowing into Newark Bay through Kill van Kull and Arthur Kill supply little oxygen for the purifying processes taking place in the Bay. It was found from an examination of the reports of the Metropolitan Sewerage Commission that from 106 analyses of flood currents and 104 analyses of ebb currents in Kill van Kull during 1911, 1912, and 1913, the excess of oxygen in the flood currents over that in the ebb currents was only 0.10 cu. cm. per liter, or less than 1.5% of the saturation value.

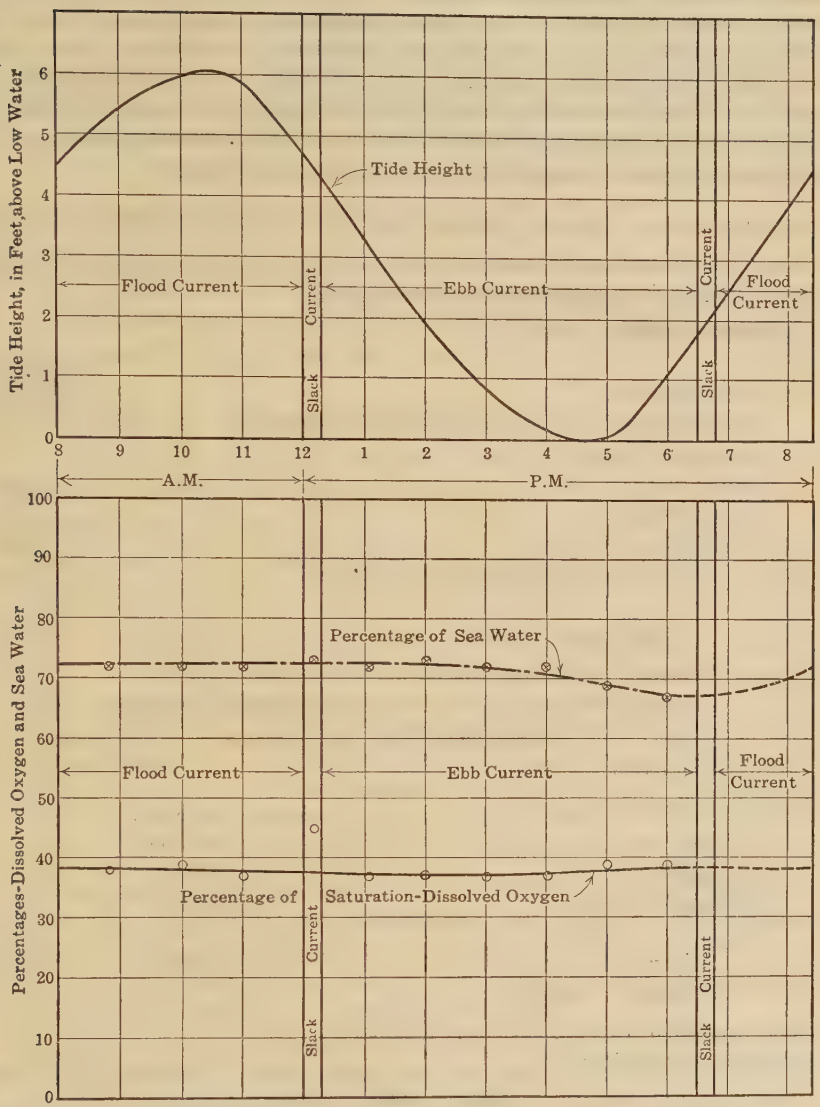
Analyses made by the writer, during 1920, covering a tidal cycle in Kill van Kull and in Arthur Kill, at Elizabeth, are shown graphically in Figs. 4 and 5. The excess oxygen carried by flood currents over that carried by the



VARIATION IN DISSOLVED OXYGEN AND SEA WATER IN THE WATERS OF KILL VAN KULL AT BERGEN POINT LIGHT, OCT. 15, 1920

FIG. 4.

ebb currents was found to be 0.16 cu. cm. per liter in the case of the Kill van Kull and 0.02 cu. cm. per liter in the case of Arthur Kill. It may be noted that the quantity of oxygen supplied to the Bay from this source is of minor importance.



VARIATION IN DISSOLVED OXYGEN AND SEA WATER IN THE WATERS OF ARTHUR KILL AT ELIZABETH, N. J. OCT. 13, 1920

FIG. 5.

Oxygen Supplied by Natural Stream Flow

Periods of low saturation in the harbor waters correspond generally with low stream flows in the inflowing rivers. Part of the dry-weather flow of the Passaic River is diverted for water supply purposes, about 81%, or about 770 sq. miles of water-shed being available for furnishing stream flow. Most of the dry-weather flow of the Hackensack River is utilized for water supply purposes. The small stream flow entering the tidal section of the river would not have an oxygen content greatly in excess of the outflowing water at the Kills, and its effect in supplying oxygen may be neglected.

Example Showing Oxygen Sources and Oxygen Demand in Newark Bay

The following approximate figures show the relative importance of the various sources of oxygen supply in Newark Bay.

Estimated Sewage Flow.—

	Millions of gallons per day.
Into Newark Bay:	
50% from Elizabeth and Joint Municipal Sewer	15.0
Bayonne	1.0
Jersey City	6.0
Staten Island	1.0
Into Hackensack River	14.0
Into Passaic River and Upper Newark Bay	120.0
Total sewage flow	157.0

Oxygen Demand of Sewage While in Bay and Rivers.—

157 000 000 gal. at 570 cu. cm. of oxygen per gallon
= 89 490 000 000 cu. cm. per day.

Oxygen Supplied by Tides.—

Volume in tidal prisms:	Cubic feet per cycles.
Newark Bay	1 070 000 000
Passaic River	195 000 000
Hackensack River	418 000 000
Total	1 683 000 000

or 3 367 000 000 cu. ft. per day.

Oxygen supplied through Arthur Kill at 0.02 cu. cm. per liter:

$$3\,367\,000\,000 \times 16\% \times 0.02 \times \frac{170}{6} = 304\,000\,000 \text{ cu. cm. per day.}$$

Oxygen supplied through Kill van Kull at 0.16 cu. cm. per liter:

$$3\,367\,000\,000 \times 84\% \times 0.16 \times \frac{170}{6} = 12\,800\,000\,000 \text{ cu. cm. per day.}$$

Total oxygen from tides..... 13 104 000 000 cu. cm. per day.

Oxygen Supplied by Rivers.—

Passaic River.—The dry-weather flow is taken at 0.2 cu. ft. per sec. per sq. mile of water-shed not used for water supply. The oxygen content is assumed to be 5.2 cu. cm. per liter at Paterson Falls and 1.8 cu. cm. per liter at Kill van Kull, or a difference of 3.4 cu. cm. per liter:

770 sq. miles at 0.2 cu. ft. per sec. $\times$ 86 400 = 13 310 000 cu. ft. per day.
13 310 000 $\times$ 3.4 $\times$ 28.32 = 1 280 000 000 cu. cm. of oxygen per day.

Hackensack River.—The entire dry-weather flow is taken for water supply:

Total oxygen from rivers = 1 280 000 000 cu. cm. per day.

Oxygen Supplied from Atmosphere.—*

Passaic River.—At 0% of saturation and 40 600 000 sq. ft. of water surface:

40 600 000 at 345 cu. cm. per square foot = 14 007 000 000 cu. cm. per day.

Hackensack River.—At from 5% to 40% of saturation:

89 000 000 sq. ft. at 166 cu. cm. per square foot..... = 14 774 000 000 cu. cm. per day.

Newark Bay.—At from 5% to 35% of saturation:

227 700 000 sq. ft. at 184 cu. cm. per square foot..... = 41 900 000 000 cu. cm. per day.

Total oxygen from air..... 70 681 000 000 cu. cm. per day.

Oxygen from All Sources.—

From tides.....	13 104 000 000 cu. cm. per day	=	15.4% of total
From rivers.....	1 280 000 000 " " " "	=	1.5% " "
From atmosphere.	70 681 000 000 " " " "	=	83.1% " "
Total	85 065 000 000 cu. cm. per day	=	100 %

The estimated oxygen demand of the entering sewage was 89 490 000 000 cu. cm. per day, 95% of which is accounted for in the apparent oxygen sources previously mentioned. The agreement is closer than might be expected from the extent of the available data. It will be noted that 83.1% of the apparent oxygen supply is estimated as coming from the air.

An attempt was made to apply this method of computation to the whole of New York Harbor. Complete data were not available, and it was necessary to make a number of approximations. Table 1 is given not because it is believed to be accurate in all its details, but because it throws an interesting light on the relative importance of the purifying agencies acting on the harbor waters.

It is recognized, of course, that the source of oxygen dissolved in water is from the air, although agreement is not general as to the rate at which this

* Absorption values are taken from Fig. 3.

takes place. Passing out from the harbor to the ocean, organic matter is diluted with such large volumes of water and spread over such a large area that the depletion in oxygen is not noticeable. The distinction between oxygen supplied from the tides and that from the air is relative only, depending on the areas considered. If the included area is taken far enough from the sources of pollution, the effects of oxygen carried by the tides are of little relative importance, as all oxygen consumed by the waters in the area is made up by absorption within the limits chosen. The area available to absorb oxygen and maintain dissolved oxygen in the waters near the sources of pollution depends chiefly on the distance and area reached by direct tidal currents and, therefore, on the volume of tidal flow passing these points. The intermixture of the direct tidal currents with waters in the areas immediately beyond their reach makes additional absorption areas available. These latter areas are polluted to a lesser degree, and, therefore, the unit rate of oxygen absorption is less. The extent of these areas will depend on the characters of the tidal currents, the tidal over-run, and other conditions difficult to define.

TABLE 1.—Absorption of Atmospheric Oxygen in New York Harbor in 1920.

Section.	Area of water surface, in million square feet.	Estimated minimum percentage of saturation of dissolved oxygen, in 1920.	Oxygen Absorbed from Air:	
			Cubic centimeters per square foot per day.	Million cubic centimeters per day.
Upper Bay	542	25	152	82 500
Hudson River (to Mt. St. Vincent.)	404	15	220	89 000
Upper East River	258	15	220	56 800
Lower East River	118	5	305	36 000
Harlem River	21.4	0	347	7 420
Kill van Kull	28.7	30	124	3 560
Newark Bay	224	15	220	49 400
Arthur Kill	116	45	59	6 830
Jamaica Bay	582	45	59	34 400
Lower Bay	3 420	80	6	20 600
Hackensack River	89.1	15	220	19 600
Passaic River	40.6	0	347	14 100
Long Island Sound, Throggs Neck-Pelham	489	80	6	2 940
Total apparent oxygen absorbed from atmosphere				423 150

In the case of New York Harbor, the waters of the Lower Bay are still well saturated with oxygen, indicating that beyond this point oxygen is absorbed at a low rate, presumably over large ocean areas. Table 1 indicates to what extent the oxygen that may be absorbed by waters, depleted to the extent of the harbor waters, compares with the estimated oxygen demand of the sewage from the Metropolitan District. The minimum saturation values found in the harbor waters are used, as it is believed that they comply with the basis of the absorption curve more than average values which include high figures due to the disturbing influence of winds, etc. Most of the saturation values

used are estimates based on the minimum values published in the 1917 Report of the Board of Estimate and Apportionment of New York City. The absorption values of atmospheric oxygen are taken from Fig. 3.

The oxygen content of the fresh water of the rivers flowing into the harbor is not materially different from that of the waters of Long Island Sound and the ocean at Sandy Hook, so that the influence of the oxygen carried by them may be neglected. If the total sewage flow of the Metropolitan District is taken as 800 000 000 gal. per day, the 423 150 cu. cm. of oxygen per day given in Table 1, represents 529 cu. cm. of atmospheric oxygen per gallon of sewage utilized by the organic matter in the sewage while it is within the harbor limits. This is about 16% greater than the estimate of the Metropolitan Sewerage Commission for the oxygen demand of New York sewage and about 7% less than that of Adeney as given previously. This seems to indicate that nearly the entire oxygen demand of the sewage in the harbor waters can be taken care of by direct absorption from the air. From this, it seems reasonable to conclude that the percentage of saturation of the harbor waters at any point will be lowered to such a value that the oxygen absorptive powers of the waters at this percentage of saturation will equal the oxygen demand of the fermenting organic matter in these waters.

A similar computation was carried out for the tidal section of the Elizabeth River. This is a small, badly polluted stream flowing into Arthur Kill. It has a water surface of 1 028 000 sq. ft., and during the four tests (Table 2), the stream flow ranged from 1.6 to 10 million gallons per day and the percentage of saturation of dissolved oxygen from 12 to 32 per cent. The polluting sewage was estimated at 800 000 gal. per day. Table 2 shows the results of the four tests.

TABLE 2.—Sources of Dissolved Oxygen in the Elizabeth River.

Date, 1920.	Oxygen Supply, in Million Cubic Centimeters per Day, from:				Percentage of estimated oxygen demand.
	Stream flow.	Tidal flow from Arthur Kill.	Absorbed from atmosphere.	Total.	
June 30	30.2	158	267	455	100
July 29	17.6	240	267	524.6	115
August 12	131.0	146	123	400	88
October 22	50.0	293	227	570	125
Average				487.4	107

Although there is some variation, due possibly to inaccuracies in the various estimates made, the results in general tend to confirm the applicability of the method of computation.

To sum up the questions discussed, the evidence seems to indicate that the oxygen demand of polluted harbor waters, when not supplied from other sources, is satisfied as far as possible by direct absorption from the air and that the dissolved oxygen in the waters will be depleted until the rate of oxygen absorption from the air equals the rate of oxygen demand of the fermenting organic

matter; that is, the area of the water surface available to absorb oxygen from the air appears to be the principal factor in controlling the condition of polluted harbor waters. The volume of water in the harbor and tidal currents, although of tremendous importance in mixing the waters and distributing the polluting material over large areas, seems to be secondary to the ability of the waters to absorb oxygen directly from the air.

Data on the oxygen demand of sewage in the waters of New York Harbor and for the construction of an absorption curve are not as complete as might be desired. It is believed, however, that the values given by the writer are sufficiently close to actual conditions to bear out at least the underlying ideas presented. With more extensive data, it might be possible that the methods indicated will permit a reasonably accurate estimate to be made of limiting quantities of sewage which the harbor will receive without the production of an active nuisance, or to predict the extent of depletion in the dissolved oxygen content for definite quantities of polluting material.

DISCUSSION

W. C. PURDY,* ESQ. (by letter).—This paper, dealing with re-oxygenation of the polluted waters of New York Harbor, is of much interest, but sufficient controlled observations do not seem to be available on which to base definite conclusions. Other factors than wind action may be responsible for great fluctuations in the oxygen content of the harbor waters. The author mentions three sources of oxygen, namely, the sea water, water of tributary streams, and the atmosphere. Granting that oxygen undoubtedly does accrue to the harbor waters from all these sources, the writer, who is a biologist, desires to call attention to another probable source.

The author makes no mention of the presence—or the absence—of aquatic plants in the waters studied. Unless the situation is totally different from that which is ordinarily found along the seaboard, especially in the more shallow waters, there is, in all probability, a considerable growth of various algæ or "sea weeds", green, brown, or red in color, also of eel-grass, and of *Ceratophyllum* or "fish-moss", commonly used in aquaria. Finally, it is practically certain that diatoms and other of the microscopic plants are present.

In the presence of sunlight, practically all these plants give off oxygen as a waste product of their metabolism. This oxygen, which is produced under water, is naturally taken up largely by the surrounding water, until, in some cases, the latter becomes supersaturated with oxygen. It would be interesting to have a detailed record of the amount of sunlight exposure on such days when large increases in oxygen content was noted between morning and afternoon observations. A close correlation of increase of dissolved oxygen and sunlight available would no doubt be observed.

The net result of this production of oxygen by submerged plants is sometimes very noticeable, not only in the oxygen content of the water, but also in the evident increase in the capacity of this water to dispose of organic matter. In the investigation of the Potomac River,† it was found that aquatic plants were largely responsible for the production of oxygen, sufficient to assist materially in the reduction of the 60 000 000 gal. of sewage emptied daily into the river by the City of Washington.

In an unpublished study of the effect of diatoms (a plant of microscopic size) on the oxygen content of water in the Ohio River, it was found that when diatoms were present to the extent of about 68 parts per million (by volume) per cubic centimeter of water, the oxygen content increased considerably more than in the same water from which the diatoms had just been removed by filtering. A similar increase was noted over the oxygen content of a third sample of the same water which contained the diatoms, but was kept in darkness, under which conditions the plants could not function properly.

It is hoped that if plants—either microscopic or large—are present in a given water, these organisms will be credited with the oxygen that they must

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† U. S. Public Health Service, Hygienic Laboratory *Bulletin No. 104*, Supt. of Documents, Washington, D. C.

necessarily contribute to the water. It would seem illogical to take account of the layer of atmosphere above the water body, only one-fifth of which is oxygen, and ignore the visible producers of practically pure oxygen, which may, to a great extent, carpet the bottom in shallow waters, and those invisible but numerous oxygen producers, the microscopic algæ, which are usually present unless the water is too deep or too turbid for the light to penetrate.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—In drawing attention to the re-aeration of water in New York Harbor by the air, especially under the influence of wind action, the author has rendered a great service. This is an important matter, and sufficient attention has not been given to it in past studies of harbor conditions.

The water in the harbor receives the sewage of the city. This sewage contains a large quantity of organic matter which is decomposing and oxidizing. Oxygen is required in order that these processes may go on. If there is enough oxygen, the processes are comparatively unobjectionable and free from the production of nuisance; if, however, the supply of oxygen at any point is overtaxed, other processes set in, that are offensive. The presence or absence of oxygen corresponds closely with conditions that are inoffensive or offensive, and it is essential, therefore, for the comfort of the communities about the harbor, that there should be enough oxygen in the water.

The air is 21% oxygen. Water dissolves only about 2% of its volume of air. Ordinarily, the water coming down the Hudson River and the sea water coming in from the ocean are saturated, or nearly saturated, with oxygen.

The quantity of water in each tide is definitely known, and the new water forming a part of it can be computed approximately. The computation is approximate, because the percentage of new water in the water coming back with the tidal flow must be estimated, and chemical analysis furnishes a fairly definite basis for this estimate. The new water entering the harbor from the sea and from the river, estimated in this way, is equal to a continuous flow of about 76 000 cu. ft. per sec.† When the rivers are in flood, the quantity of fresh water entering the harbor is greatly increased and tends to take the place of and keep out an equal volume of sea water. When the flows of the rivers are low, the sea water enters in full quantity, and almost the entire mass of water in the harbor is composed of it. As an annual average, approximately one-third of the new water is from the rivers and two-thirds from the ocean. River water carries somewhat more oxygen per unit than sea water, and cold water in winter carries more than warmer water in summer.

Without attempting refinements, the average quantity of oxygen carried by the new water, at an average temperature of 50°, is about 89 lb. per 1 000 000 gal. This is increased to 110 lb. in winter and decreased to about 65 lb. at 68°, which may be taken as summer temperature for sea water entering the harbor.

New water at the rate of 76 000 cu. ft. per sec. is equivalent to 49 200 000 000 gal. daily. The average quantity of dissolved oxygen carried by this new

* Cons. Engr. (Hazen, Whipple, and Fuller), New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 2094.

water is approximately 4 400 000 lb. per day as an annual average. The summer quantity amounts to 3 200 000 lb. per day.

The demand for oxygen is most rapid in summer, because oxidation goes on more rapidly then, and changes take place in the harbor which at other seasons are completed at remote places where other supplies of oxygen are available. The summer period, therefore, is the critical time, and summer quantities only need be considered.

The summer quantity, as estimated, is equal to 0.4 lb. daily (or 180 grammes) for each person now living in the Metropolitan District, and is sufficient to oxidize the sewage of a much larger population. In order to utilize it to its fullest extent, however, it is necessary that the sewage be completely mixed with all of it. Such a distribution is not obtained and is not possible. There are places of local concentration of sewage and, at these places, the supply of oxygen is drawn lower than elsewhere.

The secondary source of oxygen, as the author has pointed out, is the atmosphere; he thinks this is the greatest source.

When the water in the harbor is saturated with oxygen, there is no absorption of oxygen from the air by it. As soon as the water in any part of the harbor becomes less than saturated, the absorption of oxygen from the air begins, and the oxygen thus absorbed adds to the supply otherwise available. The absorption of oxygen from the air by water, other things being equal, is directly proportional to the reduction below the saturation point. If twice as much oxygen is used by the sewage in any locality, there will be twice the degree of depletion in that locality, and, with it, the rate of absorption from the air will be doubled.

Oxygen derived from the air, therefore, is greatest in amount at exactly those places where the need of it is greatest; and for this reason the value of the oxygen derived from the air is of the greatest value in keeping the harbor water sweet.

Some early studies on oxygen absorption started with the assumption that oxygen absorbed by the surface layer of water must be carried to lower layers of diffusion. If this was the case, the slowness of the diffusion process would limit the amount of oxygen that could be absorbed in this way. Fortunately, there are other means of distributing the oxygen. The tides keep the water in the harbor always in motion, and there is a mixing that is many times more powerful than diffusion in carrying the re-aerated water below the surface, and in bringing water that needs aeration from below to the surface, at which it only can be aerated. This movement due to tidal action is always going on.

Wind action is still more potent when available, but is intermittent in its action. A heavy wind brings all the water into contact with air, and results in a great increase in the rate of absorption. This increase is fairly comparable to that which takes place when a bellows is applied to a bed of coals.

The absorption of oxygen, other things being equal, is greater in shallow parts of the Bay, where the surface exposed, in proportion to volume, is greater than elsewhere, as, for instance, in Newark Bay.

How far the absorption of oxygen from the air by the exposed surface of the harbor will add to its capacity to receive and dispose of sewage, is not known. The matter is progressive; as the need of oxygen in any part of the harbor increases, the rate of absorption increases with it. Only the future, with greater populations, will disclose the limit.

In the meantime, such studies as that presented by the author, bring to notice these important fundamental actions and will tend to a better understanding of what is taking place.

WARREN R. BORST,* ESQ. (by letter).—The author's observations on the fluctuation of dissolved oxygen values in the Arthur Kill, at Elizabeth, N. J., are of special value and interest to the Borough of Richmond, especially so, as the observations made in waters adjacent to Staten Island shores seem to confirm the investigations with regard to the absorption of oxygen made at the West Brighton Sewage Experiment Station, in 1916, under the direction of the President of the Borough of Richmond. However, it would not seem possible to determine a quantitative relation between wind velocities and dissolved oxygen content, unless it was possible to determine the depth of the influence of wave action caused by the wind. From past experience and data at hand, it would seem that the depth to which the oscillation penetrates below the surface of the water depends on the violence with which the surface is disturbed.

The experimental work in the absorption of oxygen at the West Brighton Sewage Experimental Plant verified, in a large tank, the results which Adeney found to be true in glass tubes, namely, that the oxygen gravitated downward so rapidly that it was impossible to determine the rate, and, for practical purposes, in the writer's observations, was considered instantaneous for the full depth of 5½ ft. The velocity with which this diffusion or gravitation of oxygen takes place will be better appreciated by the following statement taken from the Fifth Report of the Royal Commission on Sewage Disposal of Great Britain:

"Hüfner in his work on the diffusion coefficients of gases in water met with the phenomenon of the comparatively rapid downward passage or streaming of the dissolved gas, and, in consequence, found that it was impossible to study the diffusion of gases in water when the gas was placed above the liquid, and he was compelled to employ for his experiments a thin plate of the porous mineral hydrophane to hold up a column of water in an experimental tube, and so to provide the means of introducing the gas at the bottom of the water and of studying its upward diffusion through the water.

"Experiments for determining the amount of dissolved oxygen absorbed by sewage from fresh water and sea water, as well as the rate of re-aeration of a water with its dissolved oxygen content depleted by sewage, were made in the four tanks, each of which had dimensions 25 ft. long, 6 ft. wide, and 6 ft. deep.

"These experiments may be subdivided as follows:

"1. Dissolved oxygen absorbed in dilutions of sewage with fresh water:
(*a*) Without surface agitation.
(*b*) With slight surface agitation by two electric fans.

* Asst. Engr., Borough of Richmond, West New Brighton, N. Y.

"2. Dissolved oxygen absorbed in dilution of sewage with sea water:
(*a*) Without agitation.
(*b*) With slight surface agitation by electric fans.
(*c*) With the surface agitated, making a wave, without splashing, about 4 in. high.

"3. Rate of re-aeration produced by surface agitation in a mixture of sewage and sea water containing no dissolved oxygen."

In the writer's observations, simultaneous samples, taken at each foot of depth through ¼-in. galvanized pipes projecting through the sides of the tank, in almost every instance, gave the same amount of dissolved oxygen, the amount depending solely on the violence of the agitation at the surface.

When the surface was quiet, the absorption of oxygen was small and seemed to be due to a change in temperature. The slight ripple of the surface obtained by the wind from two electric fans was sufficient to produce considerable absorption of atmospheric oxygen. A still greater absorption of oxygen was obtained by disturbing the surface with a 4-in. wave movement, produced (without splashing) by a vertical reciprocating action of a horizontally disposed furring strip located at the center and at right angles to the length of the tank.

It was noted that in fresh water, salt water, or in sewage, diluted with either fresh or salt water, the downward diffusion or gravitation of oxygen was practically instantaneous.

Neither the temperature of the water, nor the percentage of dilution of the sewage seemed to be factors in the rapidity with which this gravitation of the oxygen took place. The more violent the agitation of the surface, the greater the amount of dissolved oxygen was present. The amount of oxygen capable of being absorbed depended on the temperature, salinity, and percentage of saturation of the liquid at the beginning of the agitation.

The tanks used were 6 ft. in depth, and the observations were limited to a depth of 5½ ft. From extensive studies made at Duluth, Minn., Milwaukee, Wis., New York City, and other places, it would appear that as the effect of the wave action may extend to a depth of from 30 to 40 ft., that the depth to which this gravitation takes place would be practically the same.

In the experiments mentioned, the change of temperature of the liquid was not rapid. In many cases the temperature of the liquid in the tanks was raised to 25° cent. (77° Fahr.) by heating with exhaust steam. The temperature of the atmosphere was 10° cent. (50° Fahr.). In such cases, the temperature of the liquid did not reach the temperature of the atmosphere during the 24 hours in which the observations were made. At the bottom, the temperature was from ½° to 2° warmer than at the surface.

The rate at which the dissolved oxygen was reduced by the demand of the organic matter present in the liquid, depended on the temperature maintained in the tank. Where the temperature of the liquid was reduced from 20° cent. to 13° cent. within 24 hours, the rate at which the dissolved oxygen was reduced was practically the same as for liquid that was maintained at 20° cent. throughout the 24 hours. Where the temperature was higher than 20° cent., the rate at which the dissolved oxygen was reduced was much greater, but at no

time was the reduction greater than the oxygen obtained from the surface when agitated by a ripple ½ in. in height.

In the case of the dilution of 1 part of sewage to 20 parts of fresh water with a ½-in. ripple, it was found that after the fourth hour the rate of absorption of atmospheric oxygen was about 0.18 parts per million per hour. With salt water, the rate was 0.085 parts per million per hour.

Another observation was made with a mixture of sewage and salt water, which was devoid of oxygen. The surface was agitated with a 4-in. wave and, at the end of 4 hours, it was found that the dissolved oxygen, at all depths, had increased from 0 to 6.56 parts per million, or at the rate of 1.64 parts per million per hour. At the end of the fourth hour, the agitation was stopped, and the mixture allowed to remain quiet for 20 hours. At the end of this period, the dissolved oxygen had decreased to 4.50 parts per million, or at a rate of 0.1 parts per million per hour.

Diagrams and tables containing the detail of the studies on the absorption of oxygen were published in a "Report on Sewage Experimental Investigations at West New Brighton, Staten Island, N. Y.", by the writer, and issued by the President of the Borough of Richmond, City of New York.

The author concludes that the rapid increase of dissolved oxygen in polluted tidal waters, when roughened by the wind, is caused by the increased area exposed to the air. The experiments of 1916 at West New Brighton seem to indicate that the rapid increase of dissolved oxygen in polluted tidal waters when roughened by the wind, is due to the oscillation of the waves breaking through the layers of the water to various depths.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—The extent to which polluted waters are improved by re-aeration is a matter of much importance to towns along the shores. It requires little acquaintance with the subject to be impressed with the difficulty of arriving at any reliable quantitative solution of the matter so largely affected, as it is known to be, by disturbances of the surface and the overturning caused by the wind, currents, changes of temperature, etc.

In general, the pollution at any point in a stream depends on (*a*) the dissolved oxygen at the source of pollution; (*b*) the oxygen demand of the polluted water at that point; (*c*) the amount of re-aeration occurring between these two locations; and (*d*) the depletion of dissolved oxygen that takes place between the two points due to biological action.

The technique for the first two determinations is fairly specific and reliable, but estimates of the re-aeration and biological demand that take place in transit are subject to so many influences, and the laws governing these phenomena are either so obscure or else so lacking common endorsement, that such estimates or forecasts should be carefully weighed and expressed without too much assurance.

Temperature plays an important part in the percentage of saturation of polluted waters, so that cold-weather observations are of little value. Also, as stated by Mr. Gould, the effect of winds and currents is great, so that reliable results can only be had from the average of many tests.

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York City.

In tidal streams, there is a tendency to stratification, due to the lower specific gravity of the polluting sewage and the intrusion of the clean underrun from the ocean, so that dissolved oxygen determinations may be misleading if samples are not "representative". That this stratification may be greatly disturbed is quite apparent from cross-sections of salinity, as shown at different points in New York Harbor in the Final Report of the Metropolitan Sewerage Commission (salinity in this case being indirectly proportional to pollution). For this reason, any attempt to compute the oxygen supplied to the Upper Bay through The Narrows by the excess of the saturation on the flood over that of the ebb current, as the author has done with the Arthur Kill, would be misleading. Moreover, the constant change in conditions, due to tidal flow, introduces further complications. The dilution in a land-locked tidal basin is the ratio of the tidal prism to the volume of sewage entering during a tidal cycle. Other things being equal, a shallow bay, such as Jamaica Bay, has the advantage over a deep body of water, in that a larger proportion of its volume is discharged to the ocean with every ebb; for although the ultimate dilution is the same, at high tide a larger proportion is of new and, presumably, better oxygenated water.

The problem of re-aeration has been dealt with in an orderly way by Mr. Gould, and his results are presented in a clear concise manner. Fig. 3, on which computations of re-aeration are based, furnishes the information required in a convenient form. The writer, however, has been unable to check the results plotted from experiments of the Metropolitan Sewerage Commission, and, as the matter is of considerable importance, he hopes that the author will describe more fully, in his closure, how he arrived at these and his original experimental data. This is particularly desirable as the values he gives for re-aeration are considerably in excess of some others that have been published.

The absorption of oxygen per unit of surface is a most convenient figure to use, but in looking over the results of various experimenters it has been difficult or impossible to interpret them in this form without assumptions which do not seem to have sufficient sanction.

With regard to the rôle played by absorption from the air, E. B. Phelps, Affiliate, Am. Soc. C. E., states*:

"With the deep and relatively quiescent waters of New York Harbor re-aeration was found to be almost a negligible factor, the capacity of such a body of water to oxidize pollution is practically limited by the amount of available oxygen brought in from the ocean with each tide.

"In rivers the conditions are reversed, the initial volume of available oxygen is small compared with the total capacity of the stream to oxidize pollution. Re-aeration in rivers is the chief factor."

Adeney† also states, "when the atmosphere is calm and saturated with moisture, the rate of re-aeration will be practically at zero", but he points out further that this rule does not apply to other conditions and that "the rate of re-aeration must therefore be an extremely variable quantity".

* Reprint No. 214, Public Health Reports.

† Report, Metropolitan Sewerage Comm., 1912, p. 81.

As a matter of fact, the surface of the Upper Bay is nearly always more or less disturbed if not actually broken up by waves, and it has been shown by Messrs. Adeney, Ellery, and Borst that this increases the rate of re-aeration to a marked extent. This provides a sanction for using higher rates in practice than those determined experimentally with still water. Professor Phelps has estimated the effect of mixing due to vessels, wind, and tide, on re-aeration in New York Harbor, but, after all, the writer thinks it will be admitted that little is known about the actual rate of re-aeration in a specific case, for as yet there is no reliable measure of the influence of wind or humidity, either of which may safely be assumed to overshadow any rates for still water.

Dr. Hugh S. Cumming, of the U. S. Public Health Service (now Surgeon-General), found* in September, 1913, that in flowing from Washington to Fort Foote, in five days, the dissolved oxygen of the Potomac River was reduced from 8.13 to 4.88 parts per million. The oxygen demand of Washington sewage, however, as determined for that month, was 7.14 parts per million, which would have left only 0.99 parts per million on reaching Fort Foote. The remainder of 3.89 parts per million was made up by re-aeration.

A matter which is usually not considered, but which may have an important bearing on the oxygen content of a stream, is the regeneration of this gas by plant life. The plankton that abound in the tidal flats of the Potomac River have been estimated to furnish 17.7 lb. per acre daily of oxygen to the waters, under favorable conditions. It was computed by Mr. W. C. Purdy, in connection with the investigation of Dr. Cumming referred to, that with an oxygen demand of 300 parts per million, 1 000 000 gal. of Washington's sewage would require 2 500 lb. of oxygen for its decomposition, so that this could be accomplished by the plankton of 141 acres of tidal flats.

"In other words, this flat will provide sufficient oxygen for the first 24 hours' natural purification of nearly 30 million gallons of Washington's sewage * * *. Purely physical re-aeration * * * is certainly not over 33 per cent. of the total effect, and if proper allowance could be made for the residual biological action upon cloudy days, perhaps 6 per cent. of full sunshine, this figure would be materially reduced."

Although re-aeration by plankton is seen to be a factor of importance in locations similar to the one cited, it was pointed out by the late Emil Kuichling, M. Am. Soc. C. E., in his testimony in the Passaic Valley Sewer Case, that in the transition from fresh-water to sea-water conditions, or *vice versa,* these minute forms of vegetable life do not prosper, and it is probable that their influence in providing oxygen to such tidal bodies of water as are found in New York Harbor is not great.

Mr. Gould has estimated the quantity of dissolved oxygen available for the oxidation of the polluted waters of Newark Bay and indicated how this method may be applied to New York Harbor. Such computations aid in arriving at fair ideas of relative values and are generally illuminating, but much more definite knowledge of the influence of salinity, depth, wind, and humidity is necessary in questions of re-aeration, and regarding the draft on this supply by decomposing sludge beds, in order to predict results with con-

* U. S. Public Health Service, Hygienic Laboratory *Bulletin No. 104.*

fidence. Adeney, for instance, finds a rate for re-aeration in sea water 2.7 times that in fresh water, with an increase proportional to the depth to at least 6 ft., but leaves one in doubt as to greater depths, whereas Phelps makes the rate nearly inversely as to depth and directly proportional to the depletion, with corrections for the effect of mixing. As to the effect of sludge deposits, also, there is a wide divergence of opinion.

Taking Adeney's results, as applied to the Upper Bay during average August conditions, as well as can be told, and 1920 populations tributary to the Bay, the supply and demand of oxygen may be compiled as follows, ignoring, for the time, any demand by the underlying beds of sludge.

Population.—The population tributary to New York Upper Bay, assuming that sewage entering the Hudson River above Yonkers, N. Y., is oxidized before its arrival, is as follows:

New York City	5 231 100
Westchester County	182 600
New Jersey	1 671 200
Total	7 084 900

Water Areas.—

Hudson River below Mt. St. Vincent	421.2	million	sq.	ft.
Harlem River	21.5	"	"	"
Upper East River	280.2	"	"	"
Lower " "	116.8	"	"	"
Upper Bay	543.9	"	"	"
Kill van Kull	28.7	"	"	"
Newark Bay	224.	"	"	"

Stream Flow in August, per Tidal Cycle of 12 Lunar Hours.—

Hudson River	572 742 000	cu.	ft.
East River, net ebb	100 000 000	"	"
The Narrows, new sea water, entering on the flood	1 520 000 000	"	"

The quantity of dissolved oxygen brought into New York Harbor by streams and utilized in oxidation of sewage, is given in Table 3.

TABLE 3.

Stream.	Reduction of saturation.	Dissolved Oxygen:	
		In parts per million.	In pounds per day.
Hudson River	80% to 37%	3.61	249 142
East River	90% to 22%	5.17	62 335
The Narrows	97% to 46%	3.76	689 106
Total	.....	....	1 000 583

Dissolved Oxygen Supplied by Re-aeration.—Adeney states this to be at the rate of 0.03 cu. cm. per liter per hour, for completely de-oxygenated fresh water, increasing in proportion to the salinity to 0.08 cu. cm. per liter for sea water, and that this holds good for depths of 6 ft. If this is arbitrarily assumed as a limit and also that the rate of re-aeration is directly proportional to the depletion found in the different bodies of water, it is found that:

Pounds of oxygen absorbed from the atmosphere in 24 hours = 0.01285 [0.03 + 0.05 (proportion of sea water × depletion of DO)],

the first coefficient being the pounds of oxygen contained in a column of water, 1 ft. square and 6 ft. high, containing 1 cu. cm. per liter.

Applying this to the problem in hand, Table 4 gives the following data.

TABLE 4.

Stream.	Percentage of sea water.	Percentage of dissolved oxygen.	Absorption coefficient, in pounds per square foot per day.	Oxygen absorbed per day, in pounds.
Hudson River	60.0	37	0.00049	206 000
Harlem "	62.3	22	0.00060	12 900
Upper East River	76.5	39	0.00052	145 600
Lower " "	74.8	22	0.00066	77 000
Newark Bay	50.0	15	0.00060	134 000
Kill van Kull	63.	30	0.00055	15 800
Upper Bay	78.6	42	0.00050	271 800
Total	....	....	.....	863 100

The using of rates of absorption observed or suggested by Phelps and Ellery would give lower resulting values, whereas if the oxygen is carried by streaming to depths below 6 ft., the result would be increased.

Oxygen Demand of Sewage.—It is estimated that in 1920 the sewage of 5 413 700 persons in New York and 1 671 200 persons in New Jersey found its way to Upper New York Bay, or a total of 7 084 900 persons.

Langdon Pearse, M. Am. Soc. C. E., from many "tests on a large sewer serving 300 000 people, with no marked industrial wastes" finds* an equivalent of 0.22 lb. of oxygen per capita required for complete oxidation. The total demand for New York Harbor, therefore, may be estimated at 0.22 × 7 084 900 = 1 558 700 lb. We have, therefore,

Dissolved oxygen supplied by entering streams	1 000 600 lb.
" " " " re-aeration	863 100 "
Total	1 863 700 lb.
Dissolved oxygen demanded of sewage	1 558 700 "
Excess supply	305 000 lb.

This would seem to indicate that, under August conditions at least, the main supply of oxygen to the Upper Bay is from the incoming streams. Also, that if the sewage was well diffused and if there was no draft on the oxygen

* See p. 451.

from underlying sludge, there would remain a balance of 304 400 lb., or one-sixth of the whole supply, to take care of further pollution. Additional proof is required, however, before a positive assertion as to true values can be made. It does not seem practicable, with the present knowledge of the subject, to estimate closely the influence of the wind and the stirring up of the water by steamships and ferries in re-aeration, or that of decomposing sludge deposits in de-aeration.

Nevertheless, such studies as that which the author has presented in his paper are most valuable aids in forming a correct opinion as to the present and future degree of pollution of a body of water, and great credit is due for the manner in which the subject has been prepared and presented.

H. W. STREETER,* Assoc. M. Am. Soc. C. E. (by letter).—This interesting paper deals with a problem which, although fairly simple in its fundamental underlying principles, is often extremely complex when an effort is made to apply these principles to a particular case. This is true not only of problems involving the re-aeration of semi-quiescent bodies of water, such as New York Harbor, but also of those which deal with the re-aeration of flowing streams, although in the latter case the factors entering into a solution of such problems are usually subject to more exact measurement. Whatever the type of the particular body of water under consideration, however, there are certain fundamental reactions which must be taken into account in determining its re-aeration capacity.

The changes which occur in the reserve dissolved oxygen content of all polluted bodies of water are governed by two reactions, each of which is progressive to a great extent and more or less independent of the other. The first of these is a progressive withdrawal of dissolved oxygen from solution in order to satisfy the oxygen demand of the oxidizible organic matter present in the water. This is essentially a bio-chemical reaction, requiring, for its progression, three elements: (*a*) oxidizible organic matter; (*b*) reserve oxygen in solution; and (*c*) oxidizing bacteria to act as carriers of the reaction.

The second reaction consists of a replenishment of the reserve oxygen supply through: (*a*) additional dilution water containing oxygen; (*b*) liberation of oxygen by certain forms of aquatic plants; and (*c*) physical solution of oxygen from the atmosphere. The addition of oxygen through the medium of dilution water is a matter of physical admixture, its relative importance in any case depending on the quantity of water added and the rate at which it becomes thoroughly mixed with the body of water under consideration. The addition of oxygen through biological re-oxygenation and atmospheric re-aeration, however, are both essentially progressive reactions, dependent, for their rates of progression, on a highly complex series of factors widely differing in the two cases. Thus, purely biological re-oxygenation, which has been referred to by Mr. Purdy as being a possible factor of great importance in the re-aeration of tidal estuaries near New York Harbor, depends on the presence of certain plants and their essential food elements, together with a highly complex series of environmental conditions favorable

* San. Engr., U. S. Public Health Service, Joliet, Ill.

to their activity. Among these, the influence of light has been emphasized by the work of Messrs. Purdy, Forbes, and others, which has shown that where biological re-oxygenation exists, its progress is greatly stimulated during daylight and greatly retarded during darkness.

Atmospheric re-aeration, however, is wholly a physical reaction, dependent primarily on the rate of solution of atmospheric oxygen at the water surface. This rate is a direct function of the proportion of oxygen in the overlying atmosphere and is likewise directly proportional to the deficit in oxygen saturation existing momentarily at the absolute surface.* If complete and absolute stagnation existed in the body of water under consideration, the quantity of oxygen absorbed directly from the atmosphere would be relatively small, since, as has been shown by Phelps,† once the extreme surface film of water became saturated with oxygen at the partial solution pressure existing under atmospheric air, the penetration of oxygen to lower strata would be dependent on diffusion, which is a slow process, once a gradient has become established. Available evidence indicates, however, that complete stagnation is practically never attained in natural bodies of water, even under conditions of extreme quiescence. Under these circumstances, the extreme saturated surface film is being constantly broken up and replaced by under-saturated particles of water, the presence of which, in contact with the atmosphere, restores instantly the solution reaction. The actual rate of solution, therefore, is governed largely by the rate of surface replacement of saturated water by partly de-saturated water. In flowing streams, where the replacement rate is extremely high, its progression is determined almost wholly by conditions influencing turbulence of flow, as extensive observations in the Ohio and Illinois Rivers, by the U. S. Public Health Service, have shown. In quiescent and semi-quiescent bodies of water, this rate, although undoubtedly lower than in flowing streams, is governed primarily by the same factor of vertical circulation, which, in streams, is termed "turbulence". The work of Sanborn‡ has indicated that wind action may be, and probably is, responsible for a large part of the variation taking place in the degree and depth of vertical mixing of quiescent bodies of water. Other things being equal, this factor is probably a far more important one in determining the re-aeration of bodies of water exposed to wind action, than the increased surface area gained by the water under such circumstances.

The point, which the writer particularly desires to emphasize, however, is that whenever a polluted body of water is exposed to progressive re-aeration, the processes of de-oxygenation and re-oxygenation proceed according to two virtually independent reactions. Whenever the rate of de-oxygenation exceeds that of re-oxygenation, the dissolved oxygen content will be progressively lowered until either a sufficient proportion of the initial oxygen demand has been satisfied or sufficient additional re-aeration capacity has been attained to

* This is merely a direct application of the law of velocity of solution derived experimentally by Noyes and Whitney, *Zeitschrift für Physik. Chemie*, Band XXIII (May, 1897), p. 689.

† Report on Pollution of New York Harbor, Board of Estimate and Apportionment, New York City, 1911.

‡ "A Theory of the Water Wave", *Transactions*, Am. Soc. C. E., Vol. LXXI (1911), p. 284.

bring about an equalization or actual reversal of the two relative rates. From this point, a recovery of reserve oxygen sets in, disturbed only by the introduction of fresh pollution or by modification of the re-aeration rate. In streams, such as the Illinois River which receives the greater part of its pollution at a point near its head-waters, the influence of changes in the balance of these two reactions on the retrogression and subsequent recovery of reserve oxygen by the river may be traced for many miles below a point of major pollution.

Interpreting the author's data from the standpoint of the foregoing principles, it is evident, first of all, that a true measure of progressive re-aeration in bodies of water similar to New York Harbor, whether from biological or atmospheric sources, cannot be obtained without taking into account both the initial and residual oxygen demand of these waters, the difference, when corrected for changes in dilution, representing the amount which has been satisfied during the particular cycle of observations. If the dilution factor finally becomes sufficiently great in the outer harbor to reduce the residual oxygen demand to a value so low that it cannot be measured, the problem is virtually indeterminate, as the base line necessary for measuring the quantity of oxygen demand actually satisfied (and, consequently, the quantity of oxygen absorbed from solution), has been destroyed. It would hardly appear, however, that this need necessarily be the case, provided sampling stations are carefully selected, with the end previously stated in view, and provided, further, that determinations of residual oxygen demand at these stations can be corrected for the demand of the diluting sea water.

A further complication in calculations of this kind lies in the importance of the time factor in all the progressive re-actions which have been noted. In a flowing stream, this factor may be determined with a fair degree of accuracy, but, in tidal bodies of water, the problem is often rendered almost hopelessly complex by the vagaries of tidal movements, accompanied in some cases by marked and sudden changes in dilution. Where the age of a given body of polluted water, as measured by the time elapsed between entering and leaving the harbor waters, is relatively long, say, 5 to 10 days, the error involved in approximate assumptions bearing on the time factor is not great, but where it is so small as to fall within limits of 2 or 3 days, the error involved in such assumptions may be considerable. The assumption made in connection with the author's calculations, judging from his use of the 48-hour oxygen demand as their basis, would appear to fall in the latter category.

In connection with the figures given by the author, relative to the 48-hour oxygen demand of New York sewage, it may be of interest to note that experiments conducted by the U. S. Public Health Service in connection with stream-pollution studies in the Ohio and Illinois Rivers have indicated that at a temperature of 25° cent., an average of approximately 44% of the total oxygen demand is satisfied in 48 hours in normal composite city sewage. This figure, which is based on experiments made on the sewage of Washington, D. C., Cincinnati, Ohio, and Chicago, Ill. (in the last-named case at the outlet of the Drainage Canal), is close to the Metropolitan Sewerage Commission's figure of 48% cited by the author, but considerably lower than the

Adeney figure given. A point of doubt concerning all figures of this kind, where they are based on small-scale laboratory incubation experiments, is as to whether a particular rate as determined in the laboratory would necessarily hold true for the same water or sewage in a natural body of water. An effort is being made, in connection with studies now being conducted by the Public Health Service in the Illinois River, to establish a basis for measuring the rate at which the oxygen demand is actually satisfied in the stream itself, applying principles which have been stated in connection with this discussion.

The curve shown in Fig. 3, defining the rate of absorption of dissolved oxygen by water at different degrees of saturation, illustrates in an interesting and practical way the relation between these two variables, as determined experimentally. Although the curve does not follow closely the law of velocity of solution previously noted, it would hardly be expected that an experimental curve of the kind would do so, in view of the interference which diffusion and vertical circulation would presumably offer to a measurement of the theoretical relation. It affords a practical basis, however, for interpreting the law under conditions closely approaching those of the experiment, and it is hoped that further work along similar lines can be undertaken. Further information regarding the methods used in experiments would be of much value to investigators in this field.

Finally, it may be worth while to emphasize the point made by Mr. Purdy namely, that in casting up a balance sheet between the oxygen demand and available sources of oxygen to satisfy such demand, the possible influence of intensive biological activities in shallow bodies of water, similar to those of the tidal flats near New York Harbor, can never be safely overlooked. Whenever a marked and fairly regular increase in dissolved oxygen is noted during the day in a particular locality, as exemplified by the author's observations in the Arthur Kill, the presence or absence of aquatic plant organisms capable of liberating oxygen may be a factor of prime importance.

The author has rendered a real service in presenting the results of his study. It is hoped that further studies of the difficult and important problem with which his paper deals, will be made in the future, in order that correlations of fundamental importance in connection with problems involving the self-purification of both streams and tidal bodies of water may be established.

George A. Soper,* M. Am. Soc. C. E. (by letter).—The writer is interested in papers which deal with the pollution of New York Harbor, for it is only through the discussion of this greatest of all practical sewage problems that engineers can come to understand the conditions which exist, and form an opinion as to the character of the works which are required to dispose properly of the sewage of the Metropolitan District. For a subject of such great importance to New York City and, in fact, to the whole country, there is a singular lack of understanding of the question, and this applies to the general public and to the Engineering Profession alike.

* Cons. Engr., New York City.

An obstacle to the proper grasp of the elements of the problem lies in the tendency to regard the harbor somewhat as a laboratory vessel or tank concerning which it is easy and safe to make general observations. As a matter of fact, the sanitary protection of the waterways in the Metropolitan District probably offers the most diverse and complicated series of engineering, chemical, and biological problems that has ever been presented. After an experience of eleven years of official connection with these problems, the writer has come to look on dogmatic, unqualified, and sweeping assertions concerning them with considerable impatience.

The author seems not to be one who is willing to confine himself to the quick and easy method of argument, but has taken data from the reports of the Metropolitan Sewerage Commission and, with information of his own collecting, has sought to advance the existing knowledge of the subject by calculations of the sources of oxygen, which can be utilized in finally disposing of the sewage. The fact that the writer cannot fully agree with him in his conclusions, does not destroy his sense of the value of the paper.

That large quantities of oxygen are absorbed from the atmosphere has long been recognized, but no one has gone as far as the author in attributing such a generous share of the renewal to this source. If "the dissolved oxygen in the waters will be depleted until the rate of oxygen absorption from the air equals the rate of oxygen demand of the fermenting organic matter", the question naturally arises, how is it that the oxygen content throughout New York Harbor is falling lower and lower, until, in some of the main divisions, it has recently been reported as absent?

In spite of its many excellences, the paper contains ambiguities and inconsistencies of statement which leave some of the author's views obscure. If these defects were remedied, it is quite possible that the writer would see no reason to differ with him.

H. W. Clark,* Esq. (by letter).—There can be little criticism of the conclusion drawn by the author in regard to surface absorption of oxygen, although the writer believes that the clean tidal water saturated with oxygen, or nearly so, coming into polluted harbors is, perhaps, of greater importance than Mr. Gould's figures seem to show. There is a tremendous absorption of oxygen by water when exposed to the air, and especially so when it is ruffled by wind. In the report of the Massachusetts State Board of Health for 1891, the greater absorption of oxygen in water and its absorption to greater depth due to wind action was discussed by the late Dr. Thomas M. Drown, in an article entitled "Amount of Dissolved Oxygen Contained in Waters of Ponds and Reservoirs at Different Depths."

Work on this subject has been done by the writer from time to time, and a discussion in regard to the absorption of oxygen and the purification of sewage due to such absorption, was given in his report to the Charles River Dam Committee in 1902, together with various experiments to show the purification of polluted water by oxygen absorption. Certain of these experiments with fresh water (90% saturated with oxygen), to which was added, in dif-

* Chf. Chemist, Massachusetts Dept. of Public Health, Boston, Mass.

ferent bottles 1, 3, 5, and 10% of strong sewage, showed the exhaustion of oxygen and the development of odors under closed conditions, whereas with surface exposure, the absorption of oxygen to replace that used in purification, was very marked. In this report, after calling attention to the changes occurring after the Charles River Tidal Estuary would be turned into a fresh-water basin, preventing the entrance of sea water saturated, or nearly so, with oxygen, and calling attention to the volume of sewage that would enter it daily, the following statement was made:

"In this last connection, it must be understood that under any condition which can exist in the basin the surface water will be continually changing. The basin lies so that it is exposed to the unrestricted sweep of the prevailing summer wind and even at times of light breeze the change will be quite rapid and hence each day there is a chance for much oxygen to be taken up by the water of the basin in this way."

Mr. E. A. Letts,* has stated that Dr. Adeney found that 10 days were required to saturate with oxygen a sheet of water with a temperature of 15° cent., 6 ft. in depth, and deprived of all atmospheric gases. In experiments made by the writer in 1912, and described in the report of the State Board of Health for that year, it was shown that quiet water in a body only 7 in. in depth required 7 days to increase from 0.38 parts per 100 000 dissolved oxygen to 0.94 parts per 100 000, or saturation at the temperature of the experiment. The increase was at the rate of 0.08 parts per day. When, however, this shallow vessel contained a small fish, an amount of oxygen equal to 0.26 parts per day was absorbed, or the amount consumed by the fish daily, as had been previously proved. This experiment covered a period of 34 days. In this case, undoubtedly the surface of the water was more or less ruffled by the movements of this fish. Recent experiments by the writer have also seemed to show that, as Adeney has stated, dissolved oxygen streams from the top to the bottom of a liquid. To prove this, a glass cylinder, 10 in. in diameter and 3 ft. deep, was filled with water polluted to such an extent that dissolved oxygen failed to be present. Methylene blue added to this water was decolorized at first, owing to the absence of oxgyen, but as oxygen was absorbed, streams of the blue dye were shown throughout the depth of the water, thus indicating the penetration of the oxygen from the surface downward. This experiment was with an absolutely quiet surface.

Gordon M. Fair,† Esq. (by letter).—In May, 1919, the writer conducted a series of experiments on re-aeration in the Laboratory of Sanitary Engineering of Harvard University, under the direction of Professor George C. Whipple, M. Am. Soc. C. E., who was then investigating the pollution of New York Harbor. These experiments contribute some interesting information to Mr. Gould's study and are discussed with the permission and at the suggestion of Professor Whipple.

Two series of tests were run at short notice and with limited equipment; the results obtained were, perhaps, somewhat crude, *conditio sine qua non*, but

* *Journal*, Royal Sanitary Inst., Vol. XXXIII, No. 1.

† Director, Bureau of Statistics and Research, Cuyahoga County Public Health Assoc., Cleveland, Ohio.

warranted an interpretation of surface aeration that seems to be substantiated by recent investigations on sewage disposal and treatment. Four large containers with a capacity of about 12 gal. were filled with Cambridge tap water and freed from oxygen (as far as this was possible) by boiling. The containers were then placed in ice water in order to cool the de-aerated liquid. In the first series of tests, the surface of the water was covered with carbonic acid gas for the purpose of minimizing oxygen absorption from the atmosphere during cooling. As a result, the water became saturated with carbon dioxide, a condition that seemed to affect the results of the first experiment. This proceeding, therefore, was not repeated in the second run, and equally good de-aeration was obtained.

After the temperature of the water had been reduced to 20° cent., and the carbonic acid gas had been blown off the surface, the water was permitted to aerate under the following conditions:

1.—Water surface undisturbed.

2.—Water surface agitated by air current from electric fan; an air velocity of 1 200 ft. per min. (13.6 miles per hour) was obtained in the first test, and a velocity of 750 ft. per min., (8.5 miles per hour) in the second test. The rippling of the water was naturally greater with the higher air velocity.

3.—Water surface agitated by waves produced by balancing the container on a round rod and moving it slowly up and down.

4.—Water surface agitated as under both Conditions (2) and (3).

Dissolved oxygen samples were taken at the beginning and at frequent intervals throughout the test, which, in some cases, extended over 2 and 3 days. The change in dissolved oxygen content permitted the computation of the rate of oxygen absorption by taking into account the surface area and capacity of the container, the time elapsing between samples, and the temperature of the water. A second series of experiments, made in substantially the same manner, gave results more consistent than those of the first series. The results of this second series, expressed in cubic centimeters of oxygen absorbed per square foot of water surface per day and standardized to a temperature of 20° cent., are shown graphically in Fig. 6.

The discrepancies between the two series of results were presumably due to the disturbing influence of the carbon dioxide on oxygen absorption during the first test. This gas probably streamed out of the liquid while the experiment was in progress and, settling over the water surface, prevented aeration whenever it was not removed by the current of air from the fan. The lower wind velocity in the second series of experiments, (750 ft. per min. against 1 200 ft. per min.), which produced only a very slight rippling of the water surface, probably also contributed to the difference in the rate of oxygen absorption by the water exposed to wind action.

In view of the crudity of these simple laboratory tests, the results obtained are best considered in their relation to each other; they should not be regarded as attempted determinations of the actual coefficient of oxygen absorption for wide stretches of wind and wave-swept waters. The absorption values of the undisturbed water, however, are in close agreement with those of the

composite curve shown by the author. Taking the author's figures as a basis for comparison, the influence of wind and wave action is manifested in the writer's first series of tests, by a high rate of oxygen absorption, equal for wave action, to twice the absorption of the undisturbed water, for wind action, to five times, and for wind and wave action combined, to fifteen or twenty times. In the second series of tests, the increase over quiescent water was twofold for wind action and almost tenfold for wave action as well as wind and wave action combined. These ratios seem to remain fairly constant, regardless of the saturation value of the water. In the first experiment, wind action accelerated the oxygen absorption to a greater degree than wave movement; in the second, wave action was practically as effective as wind and wave movement combined. As suggested previously, the presence of carbonic

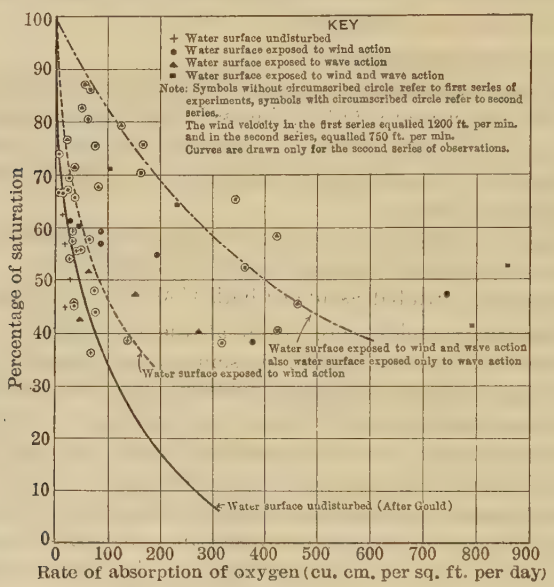
KEY
+ Water surface undisturbed
• Water surface exposed to wind action
▲ Water surface exposed to wave action
■ Water surface exposed to wind and wave action
Note: Symbols without circumscribed circle refer to first series of experiments, symbols with circumscribed circle refer to second series.
The wind velocity in the first series equalled 1200 ft. per min. and in the second series, equalled 750 ft. per min.
Curves are drawn only for the second series of observations.
Water surface exposed to wind and wave action also water surface exposed only to wave action
Water surface exposed to wind action
Water surface undisturbed (After Gould)
Percentage of saturation
Rate of absorption of oxygen (cu. cm. per sq. ft. per day)
0
10
20
30
40
50
60
70
80
90
100
0
100
200
300
400
500
600
700
800
900

FIG. 6.

acid gas may explain this difference in behavior. The results of the second experiment approached more nearly actual conditions, because normally the quantity of carbon dioxide present in surface water is altogether too small to bring about the conditions obtaining in the first test. The fact that re-aeration increased only slightly under wind action in the second test, the water being practically devoid of carbon dioxide and exposed to a weak air current that caused little disturbance of the water surface, is, therefore, quite consistent with one's expectations; likewise the fact that wave action under these circumstances was about as effective as wind and wave movement combined.

As wind seems to increase aeration chiefly through the induced wave movement, except when a heavy gas overlies the water, there are two principal factors that account for the remarkable acceleration of re-aeration produced whenever the surface of the water is disturbed, namely, the increase in surface

area as a result of wave formation, and the increase in oxygen convection induced by mixing the water and bringing new surfaces of liquid in contact with the atmosphere. The writer's calculations of the surface area of waves show an increase of only 2 to 5% for the primary surfaces of large waves. Taking into account, also, the secondary increase due to the occurrence of smaller waves on the larger ones (that is, rippling of the wave surface), it still remains difficult to explain absorption values as high as those obtained in the experiments, unless it is assumed that the wind is strong enough to produce white-caps and break the water into fine spray. Freely divided drops of spray have a large surface area, which, according to experiments made by Mr. M. C. Whipple, are capable of absorbing oxygen at rates of 100 or even 1 000 times that of quiescent water.

In the laboratory experiments under consideration, the water was not agitated sufficiently to splash and form spray; consequently, the increase in water surface by wave action does not account for the large acceleration of re-aeration. For these experiments, therefore, the factor of oxygen convection, that is, the mixing of the water, bringing new surfaces of liquid in contact with the atmosphere, seems to be the more important one. After observing, a short time ago, the operation of the new mechanical activated sludge plant at Sheffield, England, the writer is inclined to believe that this factor of convection is also the controlling force in the re-aeration of large bodies of polluted water. The Sheffield experiments clearly demonstrated that an exceedingly polluted water—for, after all, sewage is only an extreme case of water pollution—can be kept charged with oxygen by bringing changing surfaces of liquid in contact with the atmosphere. This discovery has influenced the design of the newer, English, air activated sludge plants so that this type of sewage treatment works provides for surface aeration rather than compressed air aeration, the compressed air being used to keep the sludge in suspension and bring new surfaces of liquid in contact with the atmosphere, rather than to secure aeration by bubbling.

The author asks for a thorough investigation of re-aeration, and, in the writer's opinion, the necessity of developing the newer methods of sewage treatment supports this demand.

Richard H. Gould,* Assoc. M. Am. Soc. C. E. (by letter).—Mr. Allen has asked for further description of the experiments for determining the rate of aeration of waters, as given in the paper. Those points shown on the curve in Fig. 3 and credited to the "Office of James H. Fuertes", were obtained from two series of laboratory experiments. Tap water was freed from oxygen by boiling and a known volume exposed in six small bottles for each series. The contents of a bottle was analyzed at the start of the experiment and at the end of 1 hour, 2 hours, 18 hours, 42 hours, and 8½ days. The results, calculated on the basis of oxygen absorbed per unit area of surface exposed to the air, were plotted, and a curve was drawn through the points. The two series agreed closely, although no particular refinements in laboratory methods were used. From these curves, the rate of absorption of oxygen per unit of

* Prin. Asst. Engr. with James H. Fuertes, New York City.

surface was calculated for several percentages of saturation of the water and the results were plotted as shown in Fig. 3. The points shown on Fig. 3 and credited to the Metropolitan Sewerage Commission, were computed from published analyses.* Three series of analyses for tap water and for sea water were plotted and found to agree closely with each other. A smooth curve was drawn through the points obtained from fresh water and another through those obtained from sea water, and the rate of oxygen absorption for increments of 10% of saturation was computed and plotted as shown.

From lack of knowledge of all the details connected with the various analyses, and the lack of refinement of the methods in other cases, it is not felt that the curve given in Fig. 3 is the last word in showing the true rate of absorption of atmospheric oxygen. It was thought, however, that the values shown might be reasonably near the true values. Additional experiments are desirable, and it is pleasing to note that Mr. Fair has been able to submit results of additional analyses. His values for undisturbed water surface do not deviate greatly from those shown in Fig. 3, and he is able to confirm in a quantitative way the increased aeration due to wind action and wave action, as brought out by Mr. Borst and shown by actual analyses to exist in harbor waters. It is hoped that some member of the Society will soon have occasion to carry out additional experiments showing the affect on the rate of oxygen absorption of wind and wave actions and also of humidity, which is mentioned by some as exerting considerable influence.

Mr. Purdy has called attention to the possible influence of plankton and aquatic plant life in influencing the oxygen content of harbor waters. In New York Harbor and particularly in Newark Bay, sea-weeds, grasses, and sea mosses do not seem to be present to any considerable extent. There is a continual deposition of sludge on the bottom of the Bay and on shallow flats. Without having experimental evidence of the presence, in large quantities, of diatoms and plant life capable of giving off oxygen, it would seem improbable that conditions for their development would be as favorable in Newark Bay as in the instances along the Potomac River cited by Mr. Purdy. The quantity of oxygen liberated by the plankton would be that contained in the carbon dioxide in the waters. The average quantity of carbon dioxide carried by the waters of the Passaic River in July and August has been such that if all its oxygen was liberated by the action of the plankton, it would effect the oxidation of about 7 000 000 gal. of sewage per day, or something less than 5% of the quantity estimated as flowing into Newark Bay. Additional carbon dioxide is formed by the bacterial action, utilizing the dissolved oxygen in the Bay waters. In the absence of definite knowledge of the importance of these various influences, one can only guess as to the total effect. All things considered, it would not be well, perhaps, to attribute a large part of the oxidizing effect to the re-utilization of dissolved oxygen which is made available by plankton. In a final and accurate balance of the various oxidizing influences, this question, of course, should receive due weight.

Mr. Allen and Mr. Hazen have discussed the various factors entering into the purification processes in New York Harbor and of some of the conflicting

* Report of the Metropolitan Sewerage Commission, 1914, pp. 738 *et seq.*

ideas and influences that have been current. There seems to be a general feeling that in casting a balance of the purifying influences in the harbor, the effect of the oxygen carried by the tide has not been given due weight. There are tremendous volumes of water flowing into and out of the harbor on each tidal cycle and several capable authorities seem to agree substantially on the amount of these flows. The fact that salt water works its way into the harbor and indeed, to a certain extent, as far north as Newburgh, N. Y., against the continued outward flow of fresh water from the Hudson River, shows that a large quantity of new salt water comes into the harbor daily. The water in the Upper Bay, during the summer months, is commonly 78% sea water and 22% land water. If the volume of this 22% land water is that given by Mr. Allen as the August flow of the Hudson River, the total daily flow of combined salt water and fresh water from the Lower Bay would be about 5 050 000 000 cu. ft. per day, which is not greatly in excess of the figures for August, given by Mr. Hazen.* In this connection, however, it might be well to note the statement† of H. De B. Parsons, M. Am. Soc. C. E., in discussing the land and sea water flowing into and out of the harbor, that,

"the quantity of 'new sea-water' is never a large proportion of the total quantity flowing on a flood tide. This 'new sea-water' is only new off places situated near the ocean, and is not new at such places as the Upper Bay, Fort Washington Point, Tarrytown, East River, or Newark Bay. At these places the 'new sea-water' is really water which has not been at the place on the previous tide, but comes from some other part of the harbor."

Even at The Narrows, the water entering on the flood tide is not fresh, unpolluted sea water saturated with oxygen, but from all available analyses it seems rather to be a mixture of fresh and polluted waters with its oxygen already somewhat depleted, and, no doubt, containing organic matter with a certain oxygen demand.

The full force of Mr. Allen's statement that the computation of the oxygen supply to the Upper Bay through The Narrows by the excess of the saturation by the flood over that of the ebb currents would be misleading, is not fully apparent. The Metropolitan Sewerage Commission has published in its reports several excellent cross-sections of The Narrows in which dissolved oxygen samples were taken at all depths and which seem to give fair averages of the dissolved oxygen content of the waters flowing in the currents at the particular time that these cross-sections were taken. Where so many conflicting influences are functioning, it would seem that careful and extensive oxygen determinations and their use would represent the surest method of arriving at satisfactory results.

Perhaps the most complete series of cross-sections at The Narrows, showing both the flood and ebb currents, are those taken on September 26th, 1911, the average percentage of saturation at each cross-section being plotted in Fig. 7, together with two additional series taken in July, 1913. Each point represents about fifteen analyses fairly uniformly spaced throughout the cross-section of The Narrows and should be representative. The importance of

* *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 2095.

† *Loc. cit.*, p. 2086

values throughout the entire tidal cycle for the correct determination of average conditions is indicated. The curve for September 26th, 1911, shows considerable excess oxygen carried in the flood current over the ebb current; that for July 3d, 1913, indicates little difference in the two currents, whereas that for July 24th, 1913, shows the ebb current to carry the excess of oxygen. The two series of cross-sections taken in July, 1913, do not support the proposition that a large excess of oxygen is supplied to the Upper Bay through the tides. That for September 26th, 1911, is most complete in its data in reference to the oxygen content on both currents, and presents the most favorable record indicating a large supply of oxygen through the tidal currents to the Upper Bay.

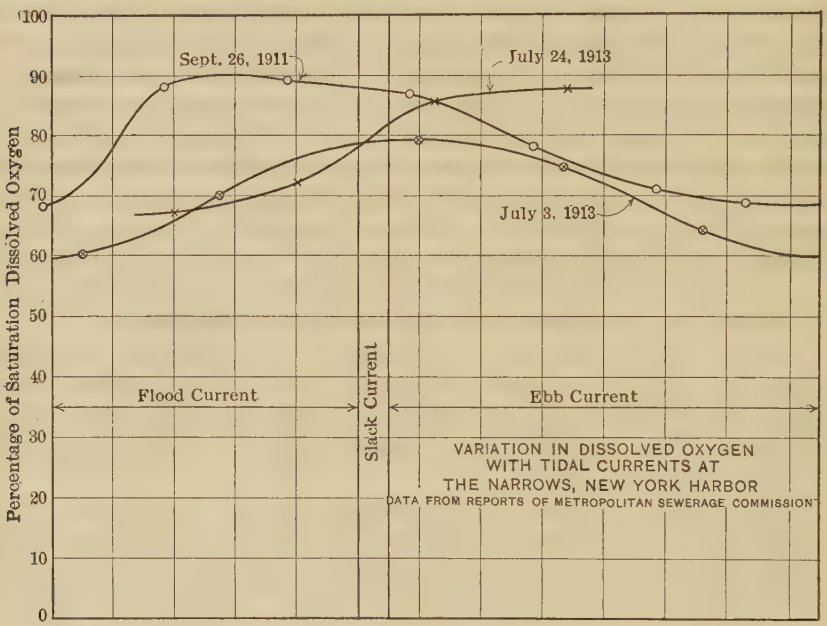

FIG. 7.

From the available data, a positive statement as to the relative importance of the oxygen supplied to the Upper Bay by the tidal currents does not seem to be justified. If the form of the curve shown for September 26th, 1911, represents average conditions at the present time, there is sound basis for the proposition that the major part of the oxygen utilized in the Upper Bay is supplied by tidal currents flowing through The Narrows. If, on the other hand, it is established that little oxygen is supplied through The Narrows, as is indicated by the average of all analyses at corresponding stages of ebb and flood currents, then the importance of other sources in supplying oxygen is emphasized.

That the same principles and values which seemed to account for existing conditions in Newark Bay, could also account for conditions in that section of the harbor included in Mr. Allen's example, if little oxygen was supplied

by the tidal currents, is shown by the example subsequently given. Mr. Allen, in his example, has apparently used average values for the summer months for the percentage of saturation of the harbor waters. The corresponding rate of absorption of atmospheric oxygen by the waters under these conditions would be influenced by a certain amount of wind and wave disturbance, and the corresponding absorption curve would lie somewhere within the limits, as shown by Mr. Fair. It does not seem unreasonable to assume that the lowest values for dissolved oxygen in the harbor would occur during the hot summer months, and when the wind and wave disturbances were at a minimum. It is questionable, perhaps, although not improbable, that minimum values would occur simultaneously in all parts of the harbor. For purposes of comparison, however, it is assumed that such would be the case, and in calculating the rates of absorption from the curve in Fig. 3, it has been assumed in the following example that the lowest recorded values for dissolved oxygen correspond to the saturation curve and are concurrent throughout the harbor. The example is as follows:

Oxygen Supplied by Streams.—

Hudson and Passaic Rivers in August:

Assumptions:

(1) Stream flow, 1 110 000 000 cu. ft. per day.

(2) Dissolved oxygen reduced from 80 to 34% at The Narrows, or 3.88 parts per million.

Total oxygen supplies = 271 000 lb. per day.

Total Currents through The Narrows and East River:

Assumed to supply no oxygen.

Oxygen Absorbed from Air.—

Assumptions:

(1) Percentage of saturation of dissolved oxygen as minimum reported.

(2) Quiescent water with absorption rates as in Fig. 3.

	Water surface, in millions of square feet.	Dissolved oxygen, percentage of saturation.	Oxygen, in cubic centimeters per square foot per day.	Oxygen, in millions of cubic centimeters per day.
Hudson River	421.2	20	184	77 500
Harlem River	21.5	0	347	7 450
Upper East River	280.2	10	263	73 700
Lower East River	116.8	5	305	35 600
Upper Bay	543.9	28	135	73 500
Kill van Kull	28.7	30	124	3 560
Newark Bay	224.0	15	220	49 300
Passaic River	40.6	0	347	14 100
Hackensack River	89.1	15	220	19 600
Total				354 310

Oxygen from All Sources.—

From streams	271 000 lb. per day
From air, 354 310 000 000 cu. cm. @ 3.15 lb.	1 117 000 " " "
Total	1 388 000 lb. per day

The total given represents 0.196 lb. of oxygen per capita for the 7 084 900 people whose sewage is estimated by Mr. Allen as flowing into the waters in question.

The total oxygen sources shown in the example given do not differ greatly from the total oxygen demand as calculated by Mr. Allen, and it is shown that the percentage of the supply which is calculated as coming from the air is somewhat greater than in his example. The example is not given as a demonstration of the correct proportion of the supply of oxygen to the different sources, but rather to emphasize again that, under conditions of oxygen depletion that have occurred in the harbor, it seems possible that a large proportion of the oxygen demand of the waters may be supplied from the air. All the assumptions made in both examples are subject to considerable variation and differences of opinion. In the summation of the oxygen sources shown by the writer for Newark Bay (page 709), the tidal currents were credited as supplying 15.4% of the oxygen. If a percentage equal to or even greatly in excess of this figure may be proved later to be correct for currents entering through The Narrows, this fact would not necessarily be inconsistent with the principles underlying the matter brought out in this paper. The physical difficulties of securing exact determinations of the various values simultaneously throughout the whole harbor are great, and until this is overcome a certain possible error may always creep into computations of this nature. Mr. Streeter has pointed out the uncertainty of fixing on a true value for the oxygen demand of organic matter in the harbor waters. Mr. Borst and Mr. Fair have shown how the rate of absorption of oxygen from the air varies with meteorological conditions. The exact quantity of oxygen carried into the harbor by the tides is also difficult to ascertain. The fundamental principles involved are not difficult to comprehend. An exact quantitative application, however, calls for additional experimental evidence of the value of the constants involved, and an extensive oxygen survey of the areas under consideration.

Mr. Soper has raised the question,

"If 'the disolved oxygen in the waters will be depleted until the rate of absorption from the air equals the rate of oxygen demand of the fermenting organic matter', * * * how is it that the oxygen content throughout the harbor is falling lower and lower until in some of the main divisions, it has recently been reported absent"?

Mr. Hazen has perhaps answered this question in his description of the purifying forces acting in the harbor, and it was thought that the matter was made clear in the original statement. The view that it was intended to bring out was that polluting matter entering the harbor waters would exert a cer-

tain demand on the oxygen dissolved therein. As the oxygen in the waters was used up the percentage of saturation would lower. With this lowering of the saturation value, the rate of absorption of oxygen from the air would increase, much as shown by the curve in Fig 3, and that this depletion of the oxygen in the water and increased oxygen absorption would continue until a saturation value of the waters was reached where the rate of absorption from the air equalled the rate of oxygen demand of the organic matter. In the Hudson River, for example, the dosage of sewage per unit of water surface area is less than that in the Harlem and East Rivers; and its dilution with oxygen-bearing waters is also less at the latter point. It would naturally follow under the assumption quoted that the percentage of saturation of waters at these two points would be different, as is actually the case.

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THE NATIONAL HOUSING PROBLEM*

A SYMPOSIUM

By Messrs. Lawson Purdy, John M. Gries, Joseph C. Wagner, B. Antrim Haldeman, Charles M. Reppert, H. Malcolm Pirnie, W. L. Stevenson, Allan Robinson, Edward M. Bassett, William H. Ham, John Ihlder, Charles Wellford Leavitt, Andrew J. Thomas, Morris Knowles, Ernest P. Goodrich, and Samuel G. Hibben.

With Discussion by Eugene W. Stern, T. Kennard Thomson, Nelson P. Lewis, John Ihlder, Henry H. Curran, Edward S. Rankin, Benjamin A. Howes, D. L. Turner, F. W. Look, William T. Lyle, and William H. Ham.

* Presented at the meetings of January 4th and 5th, 1922.

BROAD ECONOMIC PHASES OF THE HOUSING PROBLEM

By Lawson Purdy,* Esq.

It is generally recognized in the City of New York, and it is not different in most cities of the United States, that the poorest paid laborers must live in second-class houses; that it is impossible from an economic point of view under existing conditions to build houses for those persons who receive the lowest wage scale. It is not a particularly bad thing to live in a second-class house; the speaker has never lived in any other. In New York, particularly in the Borough of Manhattan, it is not desirable to live in a second-class tenement house, but many of the single-family houses, built 25 to 40 years ago, are better than the single-family houses built to-day.

The speaker has seen houses in Europe that he would like to live in, that had been built for about 200 years. It all depends on how the house was built, whether it was built for one family or for many, how it was planned, whether it is a pleasant house in which to live, regardless of when it was built.

A great many people in New York City, which is probably true of all cities, are eager to reduce the cost of building by crowding too much building on the land; and one hears constantly that the cost of housing and rents are increased by regulation designed to protect tenants in giving them a moderate amount of light and air, and adequate protection against fire. Whenever the speaker goes from New York to Boston, he wonders why insurance losses are not greater than they are. They are greater in the United States than in any other country of the world. Very likely most of the buildings deserve to be burned. If buildings were planned so that they would not burn, and so that people would have light and air, economic conditions would be better.

Engineers are doubtless familiar with conditions on the lower East Side of the Borough of Manhattan. More than 20 years ago the land was covered with tenement houses, under the law, to 70% of its area, except on corners where 90% or more was covered; but the law was not strictly administered and, often, buildings covered more than the law allowed. The least possible of light and air was left for tenants; the greatest number of people possible were crowded on an acre of ground; and the land on which such tenements were built in the lower East Side was once worth $1 000 a front foot for lots 100 ft. deep; that is, $10 per sq. ft., for land housing the poorest class of people. All that was accomplished to increase housing, by allowing that sort of thing, was that the land rose in value because it was earning such an enormous return from the poorest people. In those days, such tenements rented for not more than $5 per room per month. The landlord did nothing for the tenants except furnish the house and cold water; the conveniences were next to nothing in the old tenements.

To-day, there are men solemnly saying, and explaining that they say it as the result of long experience, that because the regulations for some such houses have been improved, the cost of human habitation has been increased in the City of New York. The speaker does not for a moment believe it.

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If it is true of the Borough of Manhattan, that no gain is made in reducing the cost of living by crowding people so densely together, by so much the more is it true of the remainder of the City of New York and of every other city of the United States. The principle is the same as that the addition of park space and open areas does not increase the cost.

Sometimes it is contended, by persons who seek the greatest return for themselves and pretend to seek the welfare of their neighbors, that government costs more and money is lost by exempting certain quasi-public institutions; it is claimed they take too much space. This was claimed in regard to the area south of Grace Church on Broadway—one of the near beauty spots of the city. The people who worship in that church had the public spirit to buy the land south of it, clear away the old buildings, and make a little open space there; and some would tax it. It is a blessing to all the surrounding territory, and all that it does is to move the business area just a few inches farther north. When open spaces are left, the population merely moves a little farther out from the most congested center. So it is with respect to regulation for light and air for human habitation. If, instead of crowding 700 people to the acre, as is done in spots on this Island, only 12 families were allowed to the acre, there would be just a little more spreading. There would be more value in the city, of lands and buildings, but that land value would not be concentrated in a few spots, to the advantage of those persons who happened to own those spots originally and to hold them. That would be the only disadvantage.

It may be truthfully said, extraordinary as it may seem, that the death rate is exceedingly low in some of these East Side blocks where the population is most congested. The reason is that in the last 20 years millions have been spent to undo the damage of congestion. The housing regulations have improved conditions; old backyard privies have been done away with; something like decent sanitary conveniences have been installed. The Health Department, which has spent millions every year, has improved the milk supply. In many ways the health of people has been conserved in spite of the conditions in which they live; and these congested centers of the Borough of Manhattan may well be compared with what looks like the attractive characteristics of mid-west cities. If, however, one examines a survey of some of these mid-west cities, it will be found that they lack a water supply; they draw their water from neighborhood wells. Sewers are lacking, and the sewage is allowed to go where each householder pleases to put it, and his water is drawn from a well on the same territory. The marvel is that they live at all. The speaker would rather live in a section which has 700 people to the acre than take his chances in one of those cities.

That, however, is not the condition desired. There should be decent housing, for housing comprises all the amenities of life. It means an adequate supply of pure water; it means proper sewers; it means that in every city no house should be permitted, which does not have a water supply, which does not stand on a street with a sewer in front of it, into which the refuse of the house must be carried. New York City has gone so far, and it is farther

than in many places, that multi-family houses are not permitted unless they are connected with an adequate sewer. A great deal more might be done. That ought to be the rule throughout the United States; and the time must come in the not distant future, when no longer will any State permit its streams to be polluted with sewage. The time must come when everywhere sewage will be disposed of in such fashion that it will not pollute the waters and will not be a menace to the health of the community.

What can be done in the near future to improve housing everywhere? First, there must be the regulation mentioned, that every house in a city must be sewered and served with water. Next, every house must be safe against fire. If wooden buildings are permitted, they must only be permitted when placed at an adequate distance from their neighbors. It is very common for the distance between houses to be no more than 10 ft.; and sometimes three-deck wooden tenement houses are seen—one can see them on the New Haven Railroad all the way from New York to Boston—that appear to stand no farther apart than 6 ft.

The speaker doubts very much whether two or more families should be allowed to live in a wooden building; but if they are, the building should be far enough from every neighbor so that there is no reasonable opportunity for fire to travel from one building to the other. Every building in which more than one family lives should be protected against the danger of loss of life by fire so that people can get out in safety if the stairs are burned. The tenement house population in the City of New York has been so far protected that since the enactment of the Tenement House Law of 1901, which is due largely to the efforts of Mr. Laurence Veiller, not one person has lost his life by the burning of such buildings.

One might say that every week, people lose their lives in tenement houses which were constructed under the law as it was prior to 1901, and many people lose their lives by fire, here and in other cities, in boarding and lodging houses. A real estate man of the speaker's acquaintance, who has had long experience, states that he has arrived at the conclusion that no building erected for one purpose should ever be permitted to be turned to any other purpose; and, in particular, no building erected for a certain kind of human occupancy should ever be permitted to be altered or changed for any other form of human occupancy, and the speaker thinks he is right.

There are many old single-family houses in this city which, if the law did not stand in the way, could be changed for the occupancy of many families. These buildings are a great menace to human life, both from fire and because of inadequate light and air. The houses were not planned for two families on every floor. They were planned for one family to occupy the whole house, and for one family it might be a fairly good house, although even then it was not safe in case of fire. It was much safer for one family, however, than it would be for several families, because there would not be so many people, and, presumably, they would be concerned about each other's safety, whereas with a lodging house in a great city, no one cares about any one but himself, and, therefore, many people are burned in lodging houses every

year. Lodging houses should be regulated by careful legislation, and no lodging house or boarding house should be of such character that the inhabitants are not as safe from fire as they are in the new law tenement houses; and they should be provided with an adequate sanitary system and adequate light and air.

All this is within the reach of the law as to buildings hereafter to be erected. As to the old ones, there is comparatively little that is politically practicable to do. The apparent economic loss of burning them down would be great. There is a large part of many cities the destruction of which would be a great blessing if the people were not burned also.

There is now the power to zone cities; although the old densely settled parts of the cities cannot be improved very much, different regulations for the outlying territory can be had, that it will be so built up that practically no house will be more than two rooms deep; so that every window shall open to the outer air on the street, or on a wider space at the rear than toward the front.

These buildings can be erected at low cost, lower than the older form of building, because the building itself can be erected in a more economical way, if it is properly planned; and when the first occupants of those new buildings have moved into better quarters, and the lower paid wage earners take possession, they can have decent houses in which to live, because they were planned rightly in the beginning. A well planned, well constructed house, 100 years old, may be better in which to live than a poorly planned house that was built yesterday. It is not intended that an old house which has been ill kept should be regarded with favor, or constructed so as to lend itself to be permeated with vermin and dirt.

Certain classes of buildings become unfit for human habitation before they have been occupied for more than two or three years. The rays of the sun are a great disinfectant, in two ways. Sunlight is believed to kill evil germs; and it is a great deal easier to keep a place clean which is bright and light than a place which is dark. With plenty of sunlight there will be more cleanliness in the old, dark houses. If the authorities decide that hereafter buildings shall be planned so as to be healthful habitations as long as they endure and are properly cared for, and that never again shall buildings be erected that are dangerous to life, health, and morals, a great deal will have been accomplished. Bad buildings bring bad morals and good buildings bring better morals and a better people.

HOUSING: BROAD ECONOMIC ASPECTS

By John M. Gries,* Esq.

In making a survey of the housing situation in the United States, it is necessary to consider present conditions, some of the causes of these conditions, and how they may be improved.

There is a general housing shortage, due to the events of the last five years, and the rate of new construction is now increasing—that much is generally known.

What, however, is the relation of the shortage to the present inventory, what are those who need homes doing to obtain them, and what is being done to meet the demand? And what provision is being made for the future? A survey of such general facts as are obtainable is not discouraging, although there should be greater conscious efforts to plan for future needs.

The houses of the United States are so widely scattered and vary so greatly in different sections, that they are rarely thought of as a whole, and their aggregate value is not often computed. An inventory, based on census figures, is the only one which can be made and, incomplete as it is, it is the only guide to the vast wealth invested in the houses of this country.

Inventory of Dwellings.—According to the Census of 1920, there are 20 697 204 dwellings in the United States. There are no reliable figures either as to cost or replacement value of these houses, but an enormous amount of the nation's wealth is in its dwellings. If the average value per dwelling is $2 000, the total value of all dwellings would be $41 394 408 000; or if the average value is $3 000, the total value would be $62 091 612 000.

Kinds of Houses.—What kind of houses are these? The number includes apartment houses, tenements, all kinds of multi-family houses, single-family houses, sod houses, freight cars, and canal boats if inhabited. Some of these houses cost more than $1 000 000, others only a few hundred dollars. Many are unfit for habitation, but still continue to be occupied.

They are constructed of various materials, stone, brick, brick veneer, stucco, tile, concrete, and wood. Few people have any idea of the ratio that the number of frame houses bears to the total. They judge largely by their own city, and answer 90% brick, or 90% frame, depending on the local conditions. Judging from estimates obtained from real estate boards, officers of building and loan associations, engineers, city officials, and others, more than 75% of all the dwellings in the United States are frame.

Judging by the statistics appearing in the press, it would seem that nearly all the houses are in the cities. For example, the building permits issued monthly and widely disseminated cover only the larger cities. Contracts awarded deal largely with the cities as far as residences are concerned. The work of both the engineer and the architect, as it relates to residential building, is largely confined to the cities. In fact, nearly every group largely eliminates rural dwellings from consideration, except the mail-order houses,

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and they probably know more about rural construction than they do about city building.

According to the Census of 1920, the number of urban dwellings was 9 484 552, and the number of rural dwellings, 11 212 652. This means that 54.2% of all dwellings are rural, and 45.8% urban. Although there are more rural dwellings than urban, there are more urban families. There are 12 803 017 urban families and 11 548 659 rural families.

The housing problem has never been considered nationally. How many of the big contractors think of the housing needs five or ten years hence? Have they considered the construction needed, the annual maintenance, and the possible economies? No, houses are not thought of nationally as one does of the United States Navy, for instance. The authorities carefully consider the needs of the Navy for ten and twenty years in the future. Even the layman thinks about the cost of maintenance. With houses, we fail to get any balance until conditions are such as to force us to act.

Need for More Homes.—As a going concern, the nation needs more homes, and during the last five years comparatively little has been done to increase the number. During the last ten years, the new buildings erected have not kept pace with the increase in population. Conditions have become worse, crowding has increased, and the cost of shelter has advanced materially. During the World War, the building of homes practically ceased, resulting in a shortage of dwellings, approximating at present, a million homes. Although people adjusted themselves temporarily to the crowded conditions, and some may accept them as normal, many will become more and more discontented. The effects will be shown in health and morals. There is much evidence of crowding in some of the large cities. A large number of single-family houses have been converted into two-family houses, and other large houses converted into tenements. Warehouses have also been changed into tenements. The one-family house cannot survive in a neighborhood in which the multi-family house has gained a foothold.

The situation will be met by the building of poorly designed houses of cheap construction, unless the cost of building is reduced materially from what it was in 1920. To build more houses, and better houses at lower cost, is what must be done, and to do this means that waste must be eliminated.

Physical Conditions of the Property.—The maintenance of residential property is most unsatisfactory. Thousands of houses are allowed to deteriorate. In some cases the neglect is deliberate, whereas, in others, the occupant or owner does not feel financially able to make the necessary improvements.

Too often he has been sold a "white elephant". He has been induced to buy beyond his ability to pay. Houses are often without adequate fire protection, with the resulting loss of many thousands. Hundreds of thousands of houses in the cities have become antiquated. They are being milked or bled dry, and allowed to deteriorate, to go into the discard, when more capital should be invested in them to keep them modern. Of course, there are also many houses which should have been torn down long ago.

Annual Building Necessary.—The number of families which should be provided with houses each year amounts to probably more than 300 000, and

considering the destruction of houses at about 80 000 each year, there is an annual need for nearly 400 000, but during the last five or six years the shortage has grown until now at least 1 000 000 homes are needed.

Whether it is a canal boat, or a sod house, every family wants a house of some kind in which to live. It is probably more essential than a hat or a pair of shoes. Every one does not want the same kind of house, and the choice is or should be guided largely by income.

What Kind of Houses Are in Demand.—No careful market analysis has been made to determine the number of houses needed by the different income groups. Little attention has been given to the income of the non-home owner. It has been assumed that he will continue to occupy the cast-off houses of the better paid. To determine the kind of houses wanted, incomes must be studied. Judging by income tax figures, the number of persons receiving an income between $1 000 and $2 000 makes up the largest single group, and that those receiving an annual income between $3 000 and $4 000 is much smaller, and so on up. This means that the market for small houses will be tremendously increased when such houses can be bought by those receiving an income between $1 500 and $2 000.

Meaning of More and Better Houses.—More and better houses mean more business, more happy families, better and more dependable citizens. Although better houses are most desirable, they cannot be bought and paid for in large numbers unless the cost is reduced.

Costs Must Be Reduced.—The houses of the nation represent an investment of billions of dollars. They are built extravagantly and scrapped recklessly.

What can be done to maintain property, to keep it in repair, to modernize it so that the maximum value can be secured out of the investment? Although some houses should have been scrapped earlier, thousands of houses are razed or scrapped long before the initial investment justified their destruction. The annual depreciation charge is too great. The waste must be taken out of the building industry, a way must be found to modernize old houses, and they must be protected by proper zoning ordinances.

Home Ownership.—As home ownership is admitted to be of the greatest economic value to a community or nation, it must be made possible for the man of average income to buy and pay for a home. At present costs, thousands of those who would like to own a home find it impossible to buy. Of the 24 351 682 homes, 12 943 604 are rented; 10 866 960 are owned by the occupants; and the tenure is unknown of 541 118. Out of the homes that are owned, 6 522 119 or 26.8% of the total are owned free of encumbrance; 4 059 593 are encumbered; 285 248 are unknown as to encumbrance. These figures show clearly that there is much room for an increase of home ownership.

"Own your own home" is a popular slogan for it has an appeal to every family living in a rented house. However, it is poor policy to induce families to buy homes for which they are unable to pay. A home ownership campaign, that merely induces a man of small income to buy a home beyond his ability to pay, is one that deserves public condemnation. The owner soon loses his

small cash deposit, and is compelled to start over again. The home ownership campaign that is really worth while is one that makes it possible for the man not only to acquire title, but to make his payments regularly. It means that every effort is made to reduce the cost of building, and to protect his property after he has bought it.

The percentage of those owning their homes varies widely in different cities. In New York City only 12.1% of the families owned their homes in 1900, and 12.7% in 1920. New York shows the lowest percentage of home ownership of all American cities.

Some cities have shown a marked increase in home ownership during the last 20 years. For example, the percentage of home ownership in Baltimore, Md., increased from 27.9% in 1900 to 46.3% in 1920. Other cities showing considerable increase in home ownership during the twenty years are: Philadelphia, Pa., increasing from 22.1 to 39.5%; Omaha, Nebr., increasing from 27.7 to 48.4%; and Des Moines, Iowa, from 38.5 to 51.1 per cent.

Some cities, however, show a decline, for example, Cambridge, Mass., shows a decline in the percentage of home ownership from 21.2% in 1900 to 17.6% in 1920; Springfield, Mass., from 33.3% in 1900 to 27.9% in 1920; and Los Angeles, Calif., from 44.1% in 1900 to 34.7% in 1920.

Zoning.—Those who would encourage home ownership should also take an active interest in seeing that property is protected. Many cities do not provide adequate zoning ordinances. The loss in property value is an economic loss running into the millions, a direct result of the failure to zone properly. Such losses can only be remedied by the action of society. There may be some ill-gotten gain accruing to some individuals by destroying property values in one section and thus forcing the people to move into another, but this is an economic loss to society, because good houses must be scrapped prematurely.

Not many years ago, complaint was almost universal that there was enormous waste in municipal government, that the conditions in some American cities were intolerable, and that action of some kind must be taken. Organizations were created, committees were appointed, newspapers and magazines were founded, and pamphlets in great numbers were issued in an effort to bring about better municipal government. The undertaking has been tremendous, and the task is not yet finished, but the improvement in spots has been encouraging.

Although more than a billion dollars are spent annually in city governments, even more money is spent in construction. The amount spent for building and construction is greater than that spent for municipal government. As no individual could possibly reform the extravagance of governments, so no individual or single group can eliminate the unnecessary waste in building. To bring about the desired reforms in the construction industry, the joint efforts of all groups is needed. The architect, engineer, contractor, sub-contractor, laborer, real estate dealer, building material dealer and producer, financier, and the public must all aid in bringing about better conditions. The cost of houses must be reduced through the elimination of waste in the entire construction industry.

What does it mean to build a large number of houses? Although the prime object is to provide shelter, many more are interested than the ones who are to occupy the houses. To indicate those who are interested, gives some idea of the breadth of the construction industry. To build the houses requires that the operations be financed. This calls for the banker, the building and loan association, second mortgage companies, and others, such as the insurance companies and the private lenders. We not only employ carpenters, bricklayers, masons, etc., in the building trades, but it means increased employment in all the plants supplying materials for construction. It not only means that the sales of producers are increased, but the sales of wholesalers and retailers as well.

This, however, is not all, for the building of new houses brings about greater activity in the furniture factories, stove foundries, carpet and rug factories, and in plants making draperies, curtains, kitchen utensils, picture mouldings, and all house furnishings.

The building of more homes means more taxable property. It does not call for more tax exempt bonds, nor, with few exceptions, is there tax exempt property as a product.

There should be a revival in building during 1922, but conditions do not warrant an enormous building program. A little more house-cleaning is needed in spots, and more efficient production with lower costs before great activity in building will prove of great economic value. Although law will do some good, the industry can do much more by a joint effort to remove restrictions and to eliminate the waste. In some cities, conditions are favorable for building, whereas, in others, local community action is necessary before building should start.

Waste in Construction Industry.—Attempts have been made to estimate the waste in the construction industry. That it is enormous is evident, and admitted by all who are acquainted with the industry. Certain practices, abuses, and customs must be changed before much can be expected. No single group is entirely at fault, nor can any single group bring about a reform. The engineer, architect, contractor, laborer, and building material dealer all know of unnecessary or expensive practices, but they alone can do but little. These groups, and also the public, must act.

Waste occurs in many places. To enumerate a few will make it evident that something should be done. Some of these places are:

1.—Lack of standardization.
2.—Absence of factory methods in construction.
3.—Unnecessary restrictions in building codes.
4.—Excessive capital required due to seasonal work.
5.—Jurisdictional disputes.
6.—Failure to schedule work.
7.—Trade agreements.
8.—Corrupt practices.
9.—High discount rates for second mortgage money.
10.—Failure to plan and zone a city properly.

Lack of Standardization.—If there is to be quantity production, there must be a greater standardization of parts in small houses. Why should there be more than four hundred different sizes and styles of window sash? Why so many different sizes and shapes of door hinges? Why so many different kinds of pipe? As long as the public insists on so many varieties, the manufacturers must be equipped to produce them. This means more capital invested in plant, more fixed charges, less production for stock. It also means that the wholesaler and the retailer must carry a heavier stock. This carrying charge must be passed on to the home owner or the renter. Some may say that such standardization will destroy initiative and individuality. The large and expensive house is still the object for all the initiative and individuality necessary.

As a rule, much more work is done on the job than is economical. Materials are poorly routed, often not at all. The carpenter often carries a piece of studding many feet, cuts it to length, and puts it into place, while in more efficiently managed places all stock is brought to him by common labor.

Unnecessary Restrictions in Building Codes.—Those who are truly interested in better housing do not want to see a general breaking down of the building codes, but they are interested in the elimination of the absurd restrictions and the unnecessary requirements. Two examples are sufficient to show the need of changes in some of the codes. In one city, an 8-in. brick wall is considered safe for a two-story house, and in a near-by city the code requires that the wall for such houses shall be 16 in. thick. In some cities, the floor load in such houses must be at least 40 lb. per sq. ft., and, in some, it must be 100 lb. per sq. ft. If an 8-in. wall is enough, why require one of 16-in.? If 40 lb. per sq. ft. is safe in some cities, why not in all cities?

Excessive Capital Required Due to Seasonal Character of Work.—Building is carried on actively for only a few months. People insist on doing their repair work in the summer. The seasonal character of the industry makes it necessary for many to make their income during a few months. Labor must earn enough during the few months to live on for twelve months. The contractor and building material dealer must do likewise. The contractor must be equipped to take care of the peak, even though part of his equipment lies idle for eight months of the year. Every establishment dependent on the construction industry, having a seasonal business, must maintain an investment necessary to take care of the peak load. Millions are invested to take care of the peak, and the home owner and renter pay for the excessive capital required, because they insist on doing all their work during the busy season. In most sections and during most years, building could just as well be fairly active during ten months of the year instead of five or six.

Jurisdictional Disputes.—Stoppage of work due to jurisdictional disputes has resulted in loss. At the Unemployment Conference called by President Harding, a resolution was adopted declaring that "stoppage of work from jurisdictional controversies in the construction industry should be permanently abandoned as detrimental to the public welfare."

Failure to Schedule Work.—Such failure might be interpreted as "waste in small scale production". A more or less efficient building practice has been

developed in the construction of large office buildings, large apartment houses, and typical factory buildings. In the erection of a large steel factory building, for example, the different pieces are marked, and the work is scheduled so that both labor and materials are on hand when and where wanted. The greater efficiency seems to prevail in the rougher types of factory buildings, decreasing as the type of building becomes higher. In complex and elaborately finished structures, the scheduling has not been so well worked out. There is little sign of efficient management in the building of small houses, except where they are built in large numbers. It is claimed that of the small houses a very high percentage is built by contractors who build less than ten houses annually. These contractors as a rule do not schedule their work well, nor does such building lend itself to efficient scheduling. They are uncertain of the arrival of materials; they are uncertain about the number of laborers they will have on a given date; nor do they know the exact order of work on the different jobs they have under way.

The designs of the buildings may also be different. Part of the houses may be on one side of the city, and part on another, and they may not be built of like materials. Building under such conditions makes it impossible to buy to advantage.

Cost of Financing Building.—Since 1920, the cost of financing building has been high. Besides a high interest rate, bonuses and commissions were common. Second mortgage notes are often discounted at from 10 to 20 per cent. Note shavers and second mortgage companies often discount paper 20 per cent. This may seem excessive, but it must be remembered that, with the fall in building costs, the second mortgage was more than ordinarily hazardous. If a small cash payment was required, it could easily happen that the second mortgage was not covered. Although the party who discounted the notes may have been justified, it is, nevertheless, a fact that such charges make the cost of houses entirely too high. The shortage of real estate mortgage money is in part due to the great increase in tax exempt securities.

Failure to Plan and Zone a City Properly.—The taxpayers spend millions annually, because cities are poorly planned and little or no attention is paid to zoning. The widening and cutting through of new streets alone costs enormous amounts. Much of this expense could have been saved had the most elementary rules of city planning been followed.

Thus far, this discussion has pointed out mainly the weak points in the construction industry. The conditions are not as bad as one might assume from the points considered, for the various groups interested in building are trying to bring about better conditions. They are in earnest, too, and are doing good work. What they need is encouragement and help. The public should do its part.

Building Codes.—One encouraging sign is the keen interest shown in the adoption of better building codes. In the summer of 1921, Herbert Hoover, M. Am. Soc. C. E., Secretary of Commerce, appointed an Advisory Committee on Building Codes, and later a Sub-Committee on Plumbing. The co-operation that the Committees received from associations, engineers, architects,

contractors, city officials, and others, was excellent, and showed a wide interest in many cities.

Zoning and City Planning.—The interest in zoning has been especially active. More than 50 cities have passed zoning ordinances, and 93 cities have zoning ordinances in preparation. Secretary Hoover appointed an Advisory Committee on Zoning in the summer of 1921, the work of which will be of great value in furthering sane zoning ordinances to protect the home owner's equity in his property.

Architects' Interest in Small Houses.—In the past, architects have taken little interest in the small house of 4, 5, or 6 rooms. The Small House Service Bureau has been organized, and endorsed by the American Institute of Architects. The organization furnishes house plans, quantity surveys or bills of materials, and architectural service for a small fee to the builder of houses of 6 rooms and less. Standard sizes are used in the houses. Interest in the elimination of varieties has increased noticeably among producers of building materials. The initiative often comes from the producers, and others often express a willingness to co-operate.

One of the most important facts is that the architect, contractor, building material producer and distributor, and labor are conscious that they are a part of a big industry and have some interests in common. As evidence of this consciousness one will find such an organization as the National Federation of Construction Industries, which includes many associations, contractors, engineers, producers of brick, tile, lumber, hardware, also retailers and wholesalers. Its aims are to make the construction industries serve the public more efficiently as a whole and by a process of education to make them harmonize their aims.

The National Congress of the Building and Construction Industry with its local chapters aims to accomplish work along the same lines. It includes all groups interested in construction. Local conferences have been strongly urged by Secretary Hoover since April, 1921.

Chambers of commerce, or groups organized for the purpose, have instituted community conferences in which an attempt has been made to secure the co-operation of all interested groups in the city. All these are concerned in eliminating bad practices, in seeing that no group absorbs the savings effected, for they know that only through a reduction in costs can building become active. They realize that volume of business is best for all, and that the public will not build unless satisfied that conditions are sound.

THE NATIONAL HOUSING PROBLEM: PLANNING AND ZONING

By Joseph C. Wagner,* M. Am. Soc. C. E.

Zoning is an essential element in comprehensive city planning, because it makes for orderliness in city building by endeavoring to place each building to its proper use, to construct it of the proper height, and, with the proper area, in its proper place on the city map. Thus, all industries are placed together, all business and commerce are likewise located, and all dwellings are similarly segregated.

Since planning in American cities has been almost entirely a matter of replanning, and existing conditions have had to be recognized, the effect of zoning in such replanning in the built-up sections of a city is to check haphazard development, continuing non-conforming use, but directing the placement of buildings in the undeveloped areas by fixing regulations to control future growth. It is only in this manner that nuisance-producing manufactures will be isolated in the least restricted areas, that industry will be separated from homes, and commercial business from industry.

The city plan with its basis in a comprehensive main street system, transit and transportation provisions, water-front development, parks, parkways, playgrounds, and open spaces, public and semi-public building sites, sewerage, water supply, and other public utilities, dependent largely for its carrying out on the exercise of the right of eminent domain, does not touch the right of private property in the use, height, and area of buildings affecting the housing problem, except by bringing into play zoning through the police power of government. Comprehensive city planning, therefore, is incomplete without zoning.

The housing problem is not so much a question of building more houses as it is one of building better houses under better conditions. It must also throw a mantle of protection about such houses, which indirectly will tend not only to standardize, but to increase their value for the specific purpose for which they were originally intended.

The promotion of the public health, safety, order, and general welfare, by the reasonable regulation of the location, size, and use of buildings, is affected by utilizing the police power for the limitation of the use of private property in the public interest, and is a legitimate use of that power.

Cities and Courts are beginning to recognize more and more the fact that, in the interest of general welfare, community power in these matters is paramount to that of the individual. Whatever is yielded from the individual for the public health, safety, and general welfare is yielded for the public or community purpose without compensation, because whatever is given by the individual to the community for the general welfare is received in the same measure protectively from the community for the same public welfare. A recognized authority on this subject, states that "a maxim of the police power is that every individual must submit to such restraint in the exercise of his

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liberty, or his rights of property, as may be required to remove or reduce the danger of the abuse of these rights on the part of those who are unskilled, careless, or unscrupulous". Zoning, therefore, is exercised for the general well-being.

To preserve the State in its power of government, the city must be preserved, and to preserve the city, the home, as well as industry, must be preserved and protected. On the home depends the nation. Zoning regulations, therefore, which separate home districts from intrusion by objectionable business, commerce, or industry, which intrusions tend detrimentally to affect morals as well as health and safety, will raise the standards of living as well as those of citizenship, and should be eagerly absorbed and written into the statute books of every far-seeing and foresighted community.

Philadelphia, Pa., with approximately 400 000 dwellings, a city of homes as well as a great industrial center, is vitally interested in a zoning law. In the new Charter Act, approved June 25th, 1919, the State granted the city the right to formulate regulations affecting the location, size, and use of buildings therein, and to make different regulations for different districts.

Acting under this authority, the Mayor transmitted to the Council, on October 13th, 1921, the draft of a proposed ordinance, prepared by the Zoning Commission, and this bill is now under consideration by the Council. After it is passed and approved by the Mayor, the regulations contained therein will become effective in the City of Philadelphia. These regulations provide for complete zoning by establishing use, height, and area districts, and make the zoning maps part and parcel of the ordinance. The law is not retro-active.

Use districts are classed as residential, commercial, industrial, and least restricted. Except in least restricted use, no tabular statement of permissive or prohibitive use is made, but each district is primarily designated for its special and particular use and uses accessory thereto.

The residential districts are designated for dwellings, and accessory uses thereto, such as professional offices, private garages, and private stables only. For the fullest protection of each district in its designated use, as well as to permit flexibility in operation, special location permits are required and granted by the Zoning Commission only after public hearing. In residential districts, buildings for educational, philanthropic, recreational, or religious use, which are in no way objectionable or detrimental to the residential character of the district, are permitted subject to special location permit. These uses include club houses, educational buildings, hospitals and sanitariums, nurseries, greenhouses, places of worship, recreation buildings, libraries, art galleries, public museums, philanthropic and eleemosynary institutions, and necessary public utilities, including community heating and lighting plants, and aviation fields. A public or community garage is prohibited in a residential district.

Commercial districts are designated for commerce, general business, and the sale of commodities, and for accessory uses which may include light industry. Uses permitted in residential districts are permitted in commercial districts; other uses are prohibited. By special location permits after a public hearing, structures or uses such as accessory uses, advertising structures, community heating plants, freight transportation and storage buildings, power-

generating plants and electric sub-stations, public garages, community garages, and public stables are permitted.

Industrial districts are designated primarily for industries, manufactories, and trades and buildings used for storage, including all processes using power generated by electricity, steam, gas, oil, petrol, or other expansive or explosive force, or chemical action. Uses permitted in commercial districts are permitted in industrial districts, but those permitted in least restricted districts are prohibited. Uses permitted in residential districts are permitted in industrial districts only by special location permits. As bearing on the housing problem, if it is essential for the general welfare that residential districts must be protected from invasion by industry, it is equally as important for the proper development of industry and that it should be free from restriction by the indiscriminate placing of stores and residences in its midst, that for the same public welfare such stores and dwellings should not be permitted alongside of industry. Therefore, in industrial districts, large areas, spotted by houses for homes, if allowed to exist along with the dirt, noise, smoke, gas, and odor of industry, will tend rapidly to deteriorate and eventually become slums. As a dwelling place, the slum is unfit for any human being.

Least restricted districts are designated for buildings and uses without restriction as to the kind of use or industry conducted therein, except as defined by other laws or ordinances, and shall include such uses as are excluded from other use districts. In this district, no dwelling is permitted other than for the use of such watchmen and caretakers necessary for the protection of the plant. The following uses permitted in this district only are here catalogued: Abattoirs and slaughter-houses; ammonia manufacturing; aniline color and dye manufacturing; asphalt manufacturing and refining; blast furnaces; cement, lime, and sulphates manufacturing; chlorine or bleaching-powder manufacturing; coal-tar products treatment; creosote and wood preserving use or manufacturing; fat, grease, lard, and tallow rendering and refining; fertilizer manufacturing; fish smoking or curing and shell grinding; garbage, dead animal, offal, or refuse reduction; gas manufacturing; glue, casein, size, and gelatine manufacturing; grist mills and grain elevators; industrial poisons and chemicals manufacturing; junk, scrap, metal, rags, waste paper, and rubber storing and baling; lampblack manufacturing; oil-cloth and linoleum manufacturing; oiled rubber or leather goods manufacturing; ore reduction; paints, white lead, shellac, varnish, and turpentine manufacturing; paper and pulp manufacturing; petroleum and products (refining and storing); potash refining; printing ink manufacturing; pyroxylin plastic manufacturing (celluloid, etc.); rubber, caoutchouc, and gutta percha manufacturing; sewage treatment works; stock yards; tanning, curing, and storing of raw hides and skins, leather, or hair; tar or water-proofing materials; wood, coal, or bone distillation or grinding; wool pulling and scouring and shoddy manufacturing.

In order that the legal rights of property may be protected where existing buildings or uses do not conform with the established district regulations, these conditions may be continued in any district as non-conforming.

The height districts are established for the purpose of regulating the heights of buildings by districts within the city, and are divided into five classes:

One times Height District.
One and one-half times Height District.
Two times Height District.
Two and one-half times Height District.
Three times Height District.

The number of times denotes the maximum height of any building at the street line in such district, indicated in terms of the street width, but in each height district a maximum building height limit is fixed, ranging from 45 ft. in a "one times" district to 150 ft. in a "three times" district. Above the maximum limit, a front setback is provided for in each district, varying from an increase in building height of 1 ft. in a "one times" district to an increase of 3 ft. in height in a "three times" district, to each foot of setback. Rear setbacks are fixed in each district as twice the ratio of the front setbacks in the same district.

Height exceptions are made, permitting the erection of appurtenances to buildings such as chimneys, stacks, fire-escape towers, skylights, ventilators, and architectural adornments, such as belfries, cupolas, minarets, pinnacles, spires and towers, and monuments, where such structures are not used for human occupancy and are appurtenant to buildings as mechanical or structural necessities.

The area districts are established for the purpose of regulating the areas and locations of buildings, and are divided into five classes, in each of which the area of an interior lot that may be encumbered is expressed in the percentage of the whole lot area. The districts are:

District A	30 per cent.
District B	50 per cent.
District C	70 per cent.
District D	90 per cent.
District E	95 per cent.

In the several districts there are mandatory directions for unencumbered areas in the shape of front, side, or rear yards and courts and for fixing the minimum dimensions of such areas.

The administration of the law is placed in the Zoning Division, Bureau of Surveys, Department of Public Works, which is the planning department of the city. Zoning certificates and occupancy permits for the erection, construction, alteration, conversion, or relocation of any building or buildings, for occupancy and use, other than residential, for every non-conforming use in residence districts, or for any subsequent change of use in a district, issue from the Zoning Division. Penalties are provided for violations.

Changes, revisions, or amendments may be made by the Zoning Commission subject to the approval of Council. Petitions for changes may be made to the Zoning Commission by the registered owners of more than one-half the total block frontage on any street between two main streets, and entitle such

petitioners to a public hearing. If, however, more than one-third of such registered owners protest against such change, the appeal for the change shall not be granted except by vote of three-fourths of the entire Zoning Commission.

In the formulation of its regulations, Philadelphia has had the advantage not only of zoning laws in other cities, but also of the experience gained in those cities in the administration and practical operation of their laws. However, each city is stamped with its own individuality, and, therefore, it is essential that regulations be made establishing the principle of zoning and the necessary attending maps with such degree of flexibility as to promote and facilitate their practical application, and also to permit free expansion as the development and evolution of the city require.

The principle of zoning is one of the fundamental elements in a wise comprehensive city plan, and will help materially to make the city not only greater, but a better place in which people may live in health, peace, and comfort.

THE RELATION OF ZONING TO THE HOUSING PROBLEM

By B. Antrim Haldeman,* Esq.

Although zoning is a comparatively new municipal activity in the United States, no movement having to deal with community development has received so much popular support wherever its meaning and intent have been made entirely clear to the people. This is undoubtedly due to the fact that it offers a maximum of protection and benefit to both public and private interests in the matter of urban development, with a minimum of expense and effort. Practically the only cost attached to the establishment and enforcement of a zoning code is that contingent to the making of the necessary surveys and district maps, the drafting of the code, and the administration of the law after its enactment. It is not too much to prophesy with confidence that, after its advantages and benefits become more generally understood, the establishment and enforcement of a zoning code will be considered as necessary in the administration and development of a city as the laying out of streets, the erection of buildings, and the construction of public utilities.

Zoning is a primary and essential element of city planning and of any thoroughly constructive and progressive program for the improvement and extension of a city, if proper consideration is had not only for the health, safety, and general welfare of its people, but also for its future progress and prosperity. All zoning codes aim at the same broad objects—the preservation of health, safety, order, and general welfare, the promotion of progress and prosperity, and the protection of the rights of property and of the rights of owners of property in such ownership and use. Zoning codes are somewhat similar to building and housing codes, but are applied in a more discriminating manner. Building and housing codes generally apply uniformly throughout a city, while under a zoning code a city is divided into districts, and the regulations may vary in different districts to provide for the particular needs of each district.

Zoning codes are established and enforced under authority of the police power of the State, and the protection and benefits derived from them are obtained without any liability for damages, as far as the use or improvement of private property may be affected by them. Police power, at least in the matter of zoning, might better be called community power, since the police have nothing to do with it. It is simply the power which the people of every State have an inherent right to exercise through proper legal channels in the interest of the public health, safety, morals, order, and the general welfare. The Courts are the agencies through which restraints are placed on the unreasonable or arbitrary exercise of this power, and any new or enlarged exercise of it, if protested, is subject to their review and approval or disapproval. It has always been zealously guarded against abuse, and its exercise has never been permitted to exceed limits reasonably necessary for the protection of the general public welfare.

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Although the Courts have placed no sharply defined limits on the exercise of the police power, they hold that it is not unlimited and that the validity of a zoning code will depend on their judgment and interpretation of the reasonableness and necessity of the protection afforded where the regulations involve a new or enlarged exercise of such power. Where it can be shown that the regulations are in the interest of the public health, safety, morals, and order, they will no doubt pass the legal test, but it is not entirely clear as to how far the police power can be extended in the interest of the general public welfare. The greatest emphasis has been placed on its use for promoting the public health, safety, morals, and order, but there are, among both lawyers and laymen, many earnest advocates of the theory that the public welfare and also the progress and prosperity of a community are almost equally important as matters of public interest which may be properly promoted through the exercise of the police power.

This view appears to be in harmony with the opinion rendered by the Supreme Court of the United States in upholding the Los Angeles ordinance in the Hadacheck case, which opinion, concurred in by a unanimous Court, states that "there must be progress and if in its march private interests are in the way they must yield to the good of the community." With this decision of the highest Court in the land as a guide, it is probable that reasonable regulations that can be clearly and convincingly shown to be in the interest of progress and prosperity, will have a fair prospect of being upheld by the Courts. This view is also supported by opinions handed down by the Courts in many cases confirming the right of municipal authorities to prohibit the erection of public garages, apartment houses, and buildings for similarly inappropriate uses in districts set apart for residential purposes.

Except in States where the Constitution confers the necessary authority on cities, it is held that an enabling act is necessary to permit the enactment and enforcement of zoning codes. During the past six years, twenty States have adopted such acts, twenty-seven cities have enacted complete codes covering the entire city, and fifty-eight cities have such codes in preparation; nineteen cities have enacted limited codes and thirty-five cities have such codes in preparation.

If one may judge by the experience of the cities that have enacted zoning codes, the establishment of use districts and regulations are the most important problems in efforts toward the zoning of cities. The Courts have looked with favor on the scheme of designating certain areas for residential purposes only, but one cannot be certain just how far one can go in differentiating residential areas for different types of dwellings, and this is important in the search for a solution of the housing problem.

In acquiring land for either development or speculation, a real estate operator considers the surroundings and the best use to which it can be put before he invests his money or prepares plans for development. He takes all the necessary measures to protect himself, and in handling high-class real estate this protection is accomplished by placing restrictions on the use of property and the kind, and even the cost, of the buildings to be erected on it.

With the foreknowledge of how the property is to be developed, the operator is in a position to lay out a proper system of streets and to provide the necessary drainage and other structures in the most economic manner, and the grantees under him can feel certain that their investments in property thus protected are safe. It is a purpose of zoning to enable the municipal authorities to determine and to have equally definite knowledge and control of the general character of the future development in each section of the city, in order that the citizens may be protected in their investments as well as in the use and enjoyment of their property. This applies not only to the home buyer and home owner, but to the manufacturer and the merchant who will have greater opportunities to expand their activities if the section in which they are located is especially adapted to, and protected for, their particular uses.

If during his brief ownership of property, it is within the power and authority of a real estate operator, to establish regulations prescribing the manner in which the property is to be developed, that will protect his grantees and their successors for all time, or even for a few years, it should be within the power of the municipal authorities, acting in the interest of, and for the protection of, all the people, to establish and enforce similar regulations. The municipal authorities who are responsible for the progress and prosperity of the city and for the health and safety of its citizens, should have at least equal rights with real estate operators in establishing regulations to protect the interests of the people.

One of the great driving forces behind the zoning movement is the desire for better homes, with all the comforts, conveniences, and contentment that the home supplies, and for the better protection of the citizen in the possession and enjoyment of the rights and privileges of citizenship. The conservation of the health and welfare of the people is one of the most vital purposes of modern government, and in no way can this be better or more easily accomplished than through the influence of the home. The influence of the home, its amenities, associations, and surroundings inevitably mould the character of the citizen for good or for ill. The ownership of his home gives to the citizen a pride of partnership in the progress and prosperity of the community and its institutions, and any measure of proven efficiency for multiplying the number of home owners and protecting their interests should command public support. Under the slogan of "own your own home", a very active campaign has been conducted, which has had for its purpose the greater encouragement of the home-owning habit among the people. This campaign has met with great success at a time when property appears to be practically at the peak of its value and when the future is more likely to see a decline in values rather than a further increase. During this campaign, many people of limited means have put all their earnings into the purchase of homes and have mortgaged their future income, and therefore, the home which they have purchased, represents practically the entire savings of their life time of work. Assuming this to be the case in thousands of instances throughout the country, it appears only fair that homes purchased and held under

such conditions should have every possible protection thrown around them to insure the permanence and stability of their value and of their desirability as homes. Under existing conditions, there is practically nothing to protect either the value or desirability of such property for home purposes, since it is quite possible that some industrial activity of a more or less objectionable nature, such as a public garage, soap factory, or boiler works, may be located immediately alongside or even surrounding it, with the result that its usefulness and value for residential purposes are ruined. Zoning would prevent such destruction of value and home interests, would protect the home owner, insure better living conditions, and probably do much to discourage the spirit of revolution and social unrest.

In almost every city, certain areas may be found on which more or less of a blight has fallen through the establishment of either business or industrial activities which have decreased the desirability of property for residential purposes, and where values cannot be re-established unless there comes a demand for additional property in that district for business and industrial purposes. Under properly established and enforced zoning laws, the conditions which obtain in these blighted districts would be prevented, and a stability of use and value would be created, which would not be disturbed unless there should be a general demand on the part of property owners for a revision of the regulations permitting a change of use.

As zoning must be considered a primary function in city planning, it should be given early consideration in the planning of a new town or the extension of an existing one. Before any streets are laid out or any buildings erected, the land included in the project, or in any probable extension of it, should be apportioned for the different uses contemplated, industrial, business, and residential, and their various types or classes. Exact district boundaries, of course, cannot be determined at this early stage of development, but the most appropriate and desirable locations for various uses can be roughly determined, leaving the precise boundaries to be established as the street system is projected and the land subdivided.

Topography and natural physical conditions will largely determine the most appropriate use for which any given area is best adapted. Industries, particularly large and heavy ones, should be assigned to level land contiguous to streams, where they exist, and where rail transportation is or can be made conveniently available. Business areas should also be preferably level or gently rolling land, centrally located with reference to the distribution of residential areas. There is a general tendency wherever zoning codes are being formulated to establish two or more classes of residential districts differentiated by the type of dwelling prescribed for each class. This differentiation contemplates two distinct classes defined as one-family dwellings and multiple dwellings and these may be further differentiated by type into two-family, row, group, apartment, or tenement houses, hotels, lodging houses, etc. The primary purpose, however, is to establish certain districts from which multiple dwellings, such as hotels, apartments, and tenements shall be ex-

cluded and the opportunity afforded for the enjoyment of real home life in quiet and peaceful surroundings.

The housing problem is directly affected by zoning for residential use. Row and group houses and other types of multiple dwellings that involve the intensive use of land can be much more economically constructed and provided with the necessary public utilities if they are erected on level or gently rolling land which requires a minimum disturbance of natural conditions and movement of material during development. One-family dwellings, or dwellings of the detached or semi-detached type, particularly where they occupy large lots, can be built more economically and with more satisfactory results on hilly or heavily rolling land than multiple dwellings. Efficient zoning would allocate land for the use to which it is best adapted by natural conditions as far as such allocation might be practicable. The load on the land as determined by the distribution and intensity of industrial, business, and residential use, directly affects the cost of constructing and maintaining all public works and public service. Property improvements and public service utilities constitute a large item in the cost of housing which is materially affected by the nature of the site and surroundings, and where the distribution and load limit are predetermined by a districting plan and zoning code, large economies in the construction of streets and other public improvements become possible.

An industrial area requires a street system quite different from that of either a business or residential area. It needs a less number and a less area of streets, but they should be wide, laid out in straight lines, and most substantially constructed for heavy traffic service. Other public service requirements, such as water supply and drainage, are also on an entirely different scale.

A business district requires a street system that will have ample capacity to permit the free and unobstructed movement of traffic. Business streets should not be excessively wide, and the areas necessary for adequate traffic service should be obtained by laying out the business districts in much smaller blocks than are appropriate to either industrial or residential districts. The greatest intensity of transportation and other public service will occur in business districts and must be provided for in planning the street system.

It is in the residential districts that the predetermination of the use of land permits of the greatest economies in property improvement, street layout, and the construction and maintenance of public works. After the system of main thoroughfares has been established, the development of intermediate streets for local use can be accomplished at little expense, as they require less property and less costly improvements and this is naturally reflected in the lower cost of a housing project, from which the ultimate purchaser, and the tenant, should receive benefit.

There is a difference of opinion as to whether industrial workers should live within walking distance of their places of employment, but whether they should or not, a zoning scheme which predetermines the areas to be used for industrial purposes and the areas where the workers will live, will permit the

organization of a transportation system that will give quick and direct service between factory and home.

The limitation of the height of buildings as contemplated by zoning does not greatly affect the housing problem, except in cities where it is customary to erect great apartment and tenement buildings. Such buildings should be excluded from districts allocated to one-family houses or dwellings limited by their type to not more than three stories. Where they are permitted, the height should be proportioned to the light and air space around them—streets, yards, and courts—so as to guarantee proper lighting and ventilation.

Area regulations are important as controlling the density of population and assuring ample open spaces adjacent to dwellings for purposes of lighting, ventilation, and family privacy. This is one phase of zoning that perhaps is more subject to scientific determination than any other, but as yet there has been no thoroughly satisfactory method of determination evolved, and it is the phase on which there is no Court decision to act as a guide.

Perhaps the greatest general benefits that may confidently be expected from zoning will come from the development of property for the purpose for which it is naturally best adapted and the guaranty of stability of use and value through the prohibition in residential districts of structures and uses that would tend to destroy values, create blighted areas, menace the health and safety of the people, and rob home life of the privacy, comfort, and charm that contribute most to the making of healthy, happy, and useful citizens and a prosperous and progressive city.

CITY PLANNING IN RELATION TO THE HOUSING PROBLEM

By Charles M. Reppert,* M. Am. Soc. C. E.

The economic and wise development of a city is dependent, to a great extent, on the successful and practical application of a well conceived city plan, complete in all its elements. The prime objectives of city planning are to provide, promote, and encourage the development of the city in such a way that good living conditions will be furnished and that business and industrial facilities will be afforded. Lack of city planning has resulted in the housing problem. Having convinced the public and municipal authorities of the wisdom and necessity for making the city plan, it remains to remove those practical and legal obstacles which operate to prevent or make difficult the application and execution of that plan.

The housing problem is due to the abominable living conditions in the congested districts of cities and to the house shortage. It follows that the standard of living conditions and the evils of congestion can only be relieved gradually by building along better lines. This, to a great extent, means the building up of the outlying and partly developed sections of the city. As far as the housing problem is concerned, it means that the major street system, the first requisite of any city plan, must be laid down with reference to communication from the home sites to the secondary centers and to the centers of industry and business, with particular reference to providing adequate and convenient arteries which will promote the building up of new residential districts; and it further means that the major street system must be legally adopted and defined so that land development may conform thereto. The location and planning of secondary streets, with proper reference to the requirements of traffic and development of land, must receive greater attention and more study in detail than is ordinarily given to the laying out of the plan of the main arteries. The development of secondary centers should receive the most careful attention, in order to encourage the movement of the home seeker from the congested districts to the newer environment, where the secondary business center should be located and opportunity afforded for neighborhood and district activities and interests.

The practical result of the execution of the city plan, confined necessarily in its earlier stages largely to city replanning and chiefly directed to the construction of new main arteries leading from the residential sections to the main business center, has been constantly to increase the load on congested traffic streets and would seem to tend to ultimate strangulation of traffic in the main business centers. This is probably due to the fact that such improvements are for the benefit of the entire city, whereas the development of secondary centers is of direct interest only to those living in or interested in that district. The conditions indicate that some of the earliest steps in executing the plan should be directed toward the development of secondary centers as a means of encouraging local home building and relieving business-section congestion.

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The planning of the rapid transit and utility systems should proceed with and form an integral part of the general city plan. The execution of the city plan will further be facilitated and made economically possible, and the growth of the city promoted, by the inclusion of other elements of a modern city plan, such as railroad, freight and passenger terminals, rail and water terminals, parks and playgrounds, harbor development, and the like. These are elements which contribute toward the payment of the bills for city planning, and this factor, in the last analysis, is most important.

The location and development of the main residential streets which might be termed the minor secondary streets, is a most important element in the city plan, inasmuch as on the wisdom and skill with which such streets are laid out depends, to a great extent, the economy with which the great mass of residential streets can be laid out and given proper outlets. The city plan, therefore, must set up the machinery which will make it possible properly to plan and build the minor secondary streets.

It would seem that the further one goes into the detail of carrying out the city plan, the more difficult the practical problems become. From the standpoint of the municipal engineer, moderately conversant with the status of the city plan, it would seem that practice has failed to keep pace with theory. "What shall be done" would seem to be pretty well understood, but "how it shall be done" is a problem still to be solved. A tie must be created between those who make the plan and those who will ultimately carry it out. Improved facilities for making it possible to carry out the city plan, so that it may attain its ends, must be provided, and these may be regarded as partly technical, in that a higher standard of professional service is required, which must be partly legislative in character.

The first step in the preparation of a city plan, and the most important element in this, in the mind of the city engineer, is the street system.

Next, it is necessary that the city plan, comprehending the major street system and its extension by minor streets, be legally defined and adopted. Unless this is done, no city plan can or will be carried out, as the uncontrolled development of the city will make it financially prohibitive. The writer was much interested in and impressed by a paper by Mr. Frank B. Williams, entitled "Enforcing the City Plan", which was presented at the meeting of the City Planning Conference at Pittsburgh, Pa., and was submitted by Mr. Williams in the discussion of the paper by Arthur S. Tuttle, M. Am. Soc. C. E. Mr. Williams defines very clearly the nature and importance of the problem of enforcing the city plan.

The advantages which the State of Pennsylvania enjoys under the so-called Act of 1913, under the terms of which mapped streets are protected from encroachment by new buildings, has been stressed in discussions of the subject. The writer's observation, from experience with attempts to enforce this Act in the City of Pittsburgh, leads him to believe that public opinion will make it impossible to rely on this statute, in its present form, in enforcing the city plan. It appears to him that the Act is confiscatory and inequitable in its operation and, unless unusual conditions obtain, it will not be carried out. The City of Pittsburgh has a City Planning Commission, operating as one of

the departments of the City government. A most excellent major street plan has been prepared by a group of citizens—the Citizens Committee on City Plan—and a zoning ordinance is under preparation. A well conceived plan for the street system is now available, but there would seem to be no possible means, under present legislation, of enforcing or legally adopting such plan. Those projects which can be carried out within a few years, will probably not be greatly affected, but necessarily the program is one which covers a much longer period for its execution. It would seem absolutely necessary that, if street location, planning, and construction, is to be carried out with proper reference to the controlling factors, or if it is to be carried out at all, the State laws should provide for district assessment, excess condemnation, and the protection of mapped streets. The necessity for these provisions is more apparent in a city where the topography is rugged and where practically the whole area which will be developed within a few years, has already been laid out in lots and streets.

The restrictions imposed by the old land subdivisions and recorded locations of streets, and the absence of necessary legislation, make it practically impossible to lay out new streets with proper regard to location, economy, and relation to topography. As a result, building operations are not encouraged.

The technical branch of city governments must be improved. A higher technical service is required, and city engineering departments must be raised to a standard where they can, at the proper point, take over the carrying out of the city plan. If the engineering department fails in its execution of the plan, the plan itself will fail. Precise surveys should be made, the city should be monumented, and reliable base maps should be prepared; otherwise the cost of the technical details will be such that the plan will fail in practice.

The municipality should have not only a city plan, but a carefully considered program for carrying it out, and this means consideration of costs and return—return in increased taxation and in service to the city and its taxpayers and residents. There is always grave danger, in view of the divergent interests of various sections of the city, that a plan will not be carried out in its logical order, with the effect that the resources of the city may be dissipated without positive results. Certain improvements will have a natural priority over others, just as the foundation of a house must be built before the superstructure is commenced, and the necessity for a city plan program will be evident, owing to the fact that it is possible to build the superstructure of the plan before its foundation. The order in which the plan is carried out is as important as the plan itself.

Properly conceived and wisely executed, the plan will pay for itself in increased valuations from taxes. The City of Pittsburgh widened one of the down-town streets—Second Avenue—from 45 ft. to 80 ft., and the increase in valuations of property abutting on this thoroughfare has been enormous and would seem to justify the improvement without much regard for the greater benefits which have accrued in relieving down-town traffic congestion. The same may be said of the so-called grading of the "Hump district," which was carried out in 1912, and which involved the reduction in grade of some of the most important down-town streets, by as much as 15 ft.

In the writer's opinion, the greatest defect of city planning is the lack of regard for that which is financially and physically feasible. The factors of directness and capacity in many cases would seem to over-shadow that of cost and economy, and there would seem to be a disposition on the part of the city planner not to do the thing at all unless it is done in the best possible manner. The city plan, before its adoption, should be subjected to thorough review and criticism by those most familiar with construction difficulties and costs, and the final plan should be modified in such particulars as feasibility and cost dictate.

If conditions were ideal, the planning of cities would be carried out co-operatively by the property owners, realtors, engineers, and architects, under the direction of one skilled in the broad problems of city planning. Such, however, is not the case, as each element interested carries out its work independently and without much regard for the other. The war-time housing activities of the Government were the nearest approach to such an ideal arrangement. The writer believes that much can be done by securing co-operative action between the city on one hand and real estate and property owners on the other, and that the establishment of some kind of a board having jurisdiction, with proper facilities for appeal, over property subdivision and the laying out and legal location of mapped streets, is entirely feasible. Such a board was suggested by Mr. Williams in the paper previously mentioned; and after a review of such various methods as have come to the writer's attention, it would seem that the future of the enforcement of the city plan lies in the general direction indicated by Mr. Williams. In any event, when a municipality takes action, whereby the property owner is prevented from enjoying the use of his property, there should be some commitment for compensation. Relief from taxation, temporary occupancy under restrictions, limitation of the period of prohibition of building on a mapped street, a commitment by the municipality to carry out the actual improvement within a given period, and such other provisions as may be devised to protect the equity of the property owner and enable the enforcement of the city plan, will have to be considered.

It is apparent that the problem of city planning is not so much concerned about the technical problem itself, as to the manner in which it can be carried out, and that the solution of the housing problem will be delayed for many valid reasons until some one discovers how the new vision of city planning can be carried out in an economic and practical manner.

The most important immediate contribution that can be made, would be to summarize the situation, define the problem, indicate various methods of solution, and formulate model State and municipal laws directed toward the enforcement of the city plan.

WATER AT THE TAPS

By H. Malcolm Pirnie,* Assoc. M. Am. Soc. C. E.

Present-day standards for public water supplies are high, and the majority of American cities are delivering water from their supply works that meets these standards. The water from the supply works passes through iron distribution mains to the taps of the consumers and, in many cases, deteriorates considerably so that the tap water does not exhibit so high a standard of purity as that recorded at the point of delivery. The reasons for this reversion in quality are being studied by water supply engineers and chemists and present variable and extremely interesting problems. The house builder and house-holder, however, are interested, as a rule, solely in the water as it is delivered at the tap and seldom give much weight to the reports of its condition at the supply works.

If the source of supply is known to be polluted by sewage and, in spite of reports of a clear, colorless, and odorless effluent from the purification plant, the consumer occasionally draws turbid, colored water from his faucet that gives a disagreeable odor and taste, his suspicions concerning the safety of the supply are immediately aroused. He will believe his own eyes and nose to the discredit of those of the plant superintendent and will discount the fact that the difference is due to physical and chemical changes which take place in the distribution system and pipes in his own house. If he is occasionally troubled with gastro-intestinal disturbances, he will blame the water supply and exonerate his favorite dish of oysters on the half shell or similar uncooked delicacy. Just how far his fears are groundless, however, and how far there may be some reason for them at times, is still a matter of speculation, although recent developments in the field of water purification point to the necessity for better methods of control and additional laboratory tests in the operation of some plants.

Next to the question of the safety of the water, that of its corrosive or incrusting properties is of interest to the house owner. Suppose, five or six years after a house has been built, the plaster on the ceiling becomes wet in one or more spots; the cause is traced to leaks in the piping of the hot-water supply, and the pipe is found to be eaten through by the action of the water on it. It would not satisfy the owner to know that the standard methods of testing the water at the supply works had always shown it to be alkaline in its reaction. The water delivered at his tap acted as an acid on his pipes and he would call it acid in spite of the records. An almost equally burdensome trouble occurs in the services and house plumbing in some cases where chemically softened water is supplied. The pipes gradually cake inside with deposited salts until water will no longer run through them, and then they have to be taken out and cleaned or renewed. Thus, corrosive and incrusting waters increase the cost of housing all over the country, and they have been accepted as necessary evils for many years. It would seem that inasmuch as waters exist in some localities in a state that will not seriously

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attack or incrust iron pipes, it would be possible so to treat corrosive or incrusting waters that they would be equally passive in their action. The realization of this, however, is hindered by the multiplicity of causes making methods applicable in one particular case inefficient or even detrimental to the process of purification in another case.

A soft water is always desirable from the standpoint of a house-holder. It requires less soap than hard water, is much more satisfactory for cooking, and makes it possible to wash clothes clean with less rubbing and, consequently, less wear. A soft water, however, is often more corrosive than a hard water, and this is particularly true when chemical treatment is used in the process of purification. The question of how much a soft water should be artificially hardened to reduce its corrosive properties has not been answered in general. Untreated soft waters are often furnished without any attempt to adjust their chemical properties so as to reduce their tendencies to attack the pipes, and attention has been directed toward the installation of brass pipes in houses in place of the usual galvanized steel pipes. It is obvious that this method of solving the problem increases considerably the first cost of building, and it is doubtful whether it is the proper solution. Iron must still be used in the distribution system, and a corrosive water will deteriorate in quality while passing through it. On the other hand, if the water is treated to make it passive before it enters the distribution mains, it will be delivered at the tap in much better condition, and the necessity for brass house pipes will disappear.

Besides the sanitary quality and physical attractiveness, the corrosive or incrusting tendencies, and the degree of hardness of a tap water, the house-owner is interested in the service pressure. This is dependent on three factors, namely, the pressure at which the water is delivered to the system, the relative elevations, and the adequacy of the distribution pipes. A small distribution system should be designed for fire protection and, therefore, should have a capacity much greater than that required for the ordinary consumption of water. If the pressure drops considerably at the service during the usual hours of maximum consumption, it is certain that the street mains are inadequate for proper fire protection and should be reinforced. The question of proper service pressure and fire protection is a simple problem in hydraulics and, therefore, can be expressed in figures both as to size of pipes and as to cost per service. The cost will vary according to local conditions, but will be less per service where the houses are near each other than where they are spaced far apart.

It is a fact that some waters do not change appreciably in passing through distribution systems, whereas others do change, and a comparison of the chemical and bacteriological examinations of the two types of water fails to explain satisfactorily the reasons for the difference. In these circumstances, it is necessary to look for explanations along lines that would not be indicated by the present standard methods of water analysis. The character of a city water supply is actually observed by the citizens at the taps, whereas, in a great many cases, the records of its physical and sanitary properties are made from samples collected before it enters the distribution system. When

it is discovered that a tap water falls below the accepted standard of purity, it will be wise to increase the efficiency of the various steps in the purification process so that the residual impurities in the water delivered to the city may be eliminated as far as practicable.

Contrary to a common impression, water purification is not developed to such an extent that a water recently polluted can be made as pure as the best spring water. Ordinary treatment by either slow sand or mechanical filtration will not suffice to remove all the impurities resulting from heavy pollution by sewage and, in these circumstances, it would be the part of reason to multiply the lines of defense against the chance of disease-producing agencies passing on to the consumer. A good general rule would be to multiply the steps in the purification process in direct proportion to the maximum concentration of pollution in the raw water. Practically, this would mean no purification for pure spring and well waters free from iron and dissolved gases; plain mechanical or slow sand filtration for reservoir or lake waters, with scattered rural populations on the catchment areas; filtration followed by chlorination, if there are sewered towns near the point of taking; and, for the treatment of a river water in which there is an unusually high concentration of sewage and industrial wastes, there might well be used all the following: First, chlorination; second, chemical treatment to produce optimum coagulation; third, storage of sufficient length to allow the most persistent sols time to be coagulated and to equalize variations in the dose required and that applied; fourth, rapid sand filtration; fifth, aeration; sixth, slow sand filtration; and, seventh, chlorination of the effluent. On first thought, these seven steps may seem superfluous, but the speaker has reason to suspect that a plant which used all these steps, with the exception of the fourth, allowed water to pass that may have been the cause of an epidemic of diarrhea on more than one occasion, when the second step was neglected or only partly carried out. There is always danger of something going wrong with any one of these steps, so that the others should not be used alone in treating a polluted water.

It is evident that the cost of thorough purification of a polluted water is also, in general, directly proportional to the concentration of pollution and, when this is thoroughly recognized there will be a greater tendency to pass over the polluted source near at hand and to go to a source where pollution can be controlled within satisfactory limits. It is clear that it would not be expedient to go so far afield in the search for pure water that no money would be left for the promotion of other health measures, such as the anti-tuberculosis and anti-malarial campaigns. Ordinary processes of filtration supplemented by chlorination have so nearly eliminated water-borne diseases, even in cases where the source of supply is grossly polluted, that the resources of the community served with filtered water should be directed toward the reduction of other diseases for the present. Where improvements in purification can be made at little additional cost, however, they should be carried out and, wherever the margin of cost between a supply from a polluted source and that from a source relatively free from pollution is small, the decision should be in favor of the latter.

In a paper before the American Water Works Association, the speaker pointed out some relations between colloid chemistry and water filtration which seemed to indicate explanations of many phenomena in water purification. An analogy of considerable importance was shown between the protective action of organic colloids on suspensoids and the possibility of a similar protective action on suspensions of bacteria. The following quotation* is of interest in this connection. In writing of suspensions of bacteria, Taylor stated:

"They thus stand between suspensoid and emulsoid sols, and on this account have been represented as suspensoids, protected by an emulsoid sol. * * *
"When the immune serum is added to the bacteria sol, the latter becomes much more sensitive to electrolytes. * * * It would thus appear that the agglutinin in the immune serum destroyed the 'protecting part' of the bacteria sol, which thus became a suspensoid sol."

There seems to be excellent grounds for believing that bacteria protected by adsorbed organic colloids exist in polluted waters and that, if they do exist, some of them will pass the filters and chlorination process unharmed. The protecting colloids may keep them from starting colonies on gelatine or agar plates, and thus they would not be detected. Later, in the distribution system, possibly due to the action of ferric ion, the protective action may be destroyed, and samples of tap water will show the presence of bacteria when the results at the plant were negative. The acceptance of this theory makes it easy to explain the apparent return to life of bacteria after filtration and chlorination and the variable longevity of *B. Coli* and *B. Typhi* in different waters. If this is the correct explanation of these phenomena, it is of the utmost importance to reduce the residual organic matter to a minimum, at all times, in the effluents of plants treating polluted waters.

The reduction in the colloidal organic matter is quite marked in slow sand filters, if the load is not too great, and is probably due to biological processes, but the reduction is usually negligible in rapid sand filters in the absence of coagulation. Chemical treatment is necessary, therefore, for the complete removal of organic colloids, and the use of alum alone for this purpose interferes with the chemical balance of the water by making it more corrosive. The application of soluble alkalies to offset the effect of the alum is adverse to the efficiency of the alum reaction, as the hydroxyl ion peptizes the emulsoids, increasing the total surfaces which, in turn, require more alumina for coagulation. It might be said that if the water likes to eat iron, give it iron to eat before it gets at the pipes, but this method of pacifying the water has its drawbacks in cost and in difficulty of removing the dissolved iron. A new method of treating water to counteract the acidity due to the alum reaction has been the addition of powdered calcium carbonate which probably adsorbs the excess hydrogen ions together with acid cations, so that the hydrogen ion concentration in the effluent is practically the same as that in the untreated water. There is a commercial product acting on this principle—a mixture of crushed alum and powdered calcium carbonate—that is said to be giving excellent results in several plants treating soft colored waters.

* W. W. Taylor, "The Chemistry of Colloids", p. 308.

The standard methods of water analysis will not indicate what treatment must be applied to a water to reduce corrosion and, until present tests have been considerably expanded, it will be necessary to experiment with each supply before the proper method of chemical application can be found for it. It is hoped that it will be possible in the near future, through the elimination of emulsoid sols, with their tendency to protect bacteria and to carry with them adsorbed acids, and through the scientific control of the hydrogen ion concentration, to reduce greatly the corrosive properties of water supplies and to put an end to the common apparent increase in numbers of bacteria in the distribution system. If this can be accomplished, the present deteriorated condition of many tap waters will disappear. The consumer will then draw from his faucets a water approaching in quality the high standard maintained at the effluent from the filters.

The application of the large and useful field of colloid chemistry to the study of water purification is new, and it has been found that many of the reactions observed in this special chemical field are analogous to phenomena that have long been noted in water purification. Mention has been made of the action of organic colloids in protecting minute suspensions, including suspensions of bacteria, because, if this theory has been found to explain the reactions observed in colloid chemistry, there is no reason why it should not apply to the processes of water purification. It is easy to visualize the adhesion of a coating of organic matter about a bacterium and to reason from this what its protective action might be. Such a coating could easily prevent the bacterium from sticking to the sand in the bed of a filter and through soaking up chlorine could prevent the chlorine from reaching the enclosed bacterium. Furthermore, these organic particles, through their power of soaking up acids, may carry the acids through the filters to become active in contact with iron and heat in the distribution system and house piping.

It is not reasonable to expect that colloid chemistry will give the final solution of all problems confronting the water-works engineers and chemists, but it is certainly a step in advance. Modern methods of water purification have succeeded in delivering effluents of excellent physical and sanitary qualities. The largest problem before the water engineers and chemists of to-day is that of delivering to the consumers water that will be as good in appearance and apparent health properties as when it leaves the purification works.

THE BASIC PRINCIPLES OF THE PUBLIC SEWERAGE OF MUNICIPALITIES

By W. L. Stevenson,* M. Am. Soc. C. E.

The sewerage of towns is necessary to protect the public health through the prompt removal and disposal of human excreta, to provide for the comfort of the citizens, and as an important aid to municipal expansion and development.

As long as men live isolated from one another, it is generally possible to obtain drinking water from a well and to dispose of excreta and other wastes on the surface of the ground, provided proper precautions are taken. Town life, however, changes these conditions; the smaller tracts of land and the proximity of dwellings make it unsafe to obtain drinking water from a well surrounded with cesspools and privies.

With town life also comes the popular demand for a bountiful supply of safe drinking water, the desire for inside plumbing fixtures, and for fire protection, which eventually results in the installation of a public water supply, and this, in turn, creates a need for the prompt removal of the spent water supply, the sewage from dwellings, the industrial wastes from factories, and the abatement of nuisance caused by kitchen and laundry water in gutters. With paved highways comes the need for prompt underground removal of storm water. All these demands, therefore, lead to a public sewerage system.

The officials responsible for the public works of towns do not realize sufficiently that the installation of a sewer system is an engineering problem involving technical knowledge and that economy of municipal funds may be secured through the engagement of a competent sanitary engineer.

It is not uncommon to find that the sewering of a town is begun by laying, say, an 8-in. pipe down a certain street, because there are houses along it the owners of which desire sewerage facilities, and no heed is given to the relation between this particular sewer and the whole drainage area of which it ultimately will or ought to become an integral part. The result of this haphazard procedure is that when the town grows, and it becomes necessary to extend the original sewer, it may be found of insufficient capacity, or otherwise laid so that it cannot be extended, or its outlet may be at too low an elevation to be connected to a future intercepting sewer.

Its ultimate uselessness is a sheer waste of public money, which could have been avoided by the preparation of a comprehensive sewer plan prior to construction. An engineer seldom has an opportunity to prepare such a plan for an entirely unsewered town and, therefore, the first work should be to obtain complete information concerning all known existing sewers, in order that as far as possible such as are suitable may be incorporated in the comprehensive plan. The main drainage areas of the entire municipality should be determined from the topography and the street plan, and practicable and economical locations selected for the main and sub-main sewers of each area.

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For the built-up parts of the town site, the complete sewer system can be designed in detail, with fairly safe assurance that future development will be provided for and that only in case of unusual changes will relief sewers be necessary in the years to come.

For the drainage areas or parts thereof at present undeveloped and for which no definite street plan has been adopted, the tentative location of the main and sub-main sewers may generally be determined from the topography. An experienced engineer can estimate the probable future density of population and quantity of sewage per capita, and also the storm water, with sufficient accuracy to estimate the run-off and from these data determine approximate sizes and gradients of the proposed main sewers of the undeveloped areas.

In the design of the comprehensive plan, consideration should be given to the probable future admission of sewage or storm water from those parts of drainage areas of the town at present outside the municipality, for it must be remembered that flowing water does not recognize a municipal boundary line even though it crosses a drainage area. Topography not political boundaries should govern in sewer design.

In the future, annexation of such outlying territory or agreements between contiguous municipalities may make co-operative sewerage economical of public funds and in that way justify the original increase in capacity of certain sewers of the comprehensive plan.

The comprehensive plan should include at least tentative practicable alignment, sizes, and gradients of intercepting sewers, in order that the outlets of main sewers may be designed with such elevations that, without excessive costs for pumping or reconstruction, all the sewage of the town can be conveyed to a suitable site for treatment. This should be done even where it is not necessary to treat sewage at present, in which case, general outline plans sufficient to demonstrate the feasibility of sewage treatment should be prepared.

Where the town site includes areas of appreciable size of both high land and low land, consideration may be given to the design of two lines of intercepting sewers, the higher to convey sewage by gravity to the treatment works and thus reduce the volume of low-level sewage which will have to be pumped. However, this does not always prove to be economical, as sometimes the lessened rates of flow of low-level sewage, due to the diversion of the high-level sewage, may require gradients so much steeper on the low-level intercepting sewer that the cost of construction is seriously increased by the deep cut in a possibly wet trench due to proximity to a stream.

A decision must early be made in the preparation of a comprehensive plan of sewerage as to the use of the combined or separate system of sewers and although many elements enter into this problem, usually the controlling factors will be the kind and extent of existing sewers and the relative economy, as determined by comparative estimates of the cost of construction plus the capitalized annual charges of each system.

In the older towns, rain-water culverts were constructed along the beds of streams flowing through the town site and, later, utilized for the conveyance

of sewage. This practice developed into the so-called "combined system" of sewerage, in which the conduits are made of sufficient capacity to receive the run-off of storm water on the drainage area, and the sewage is also carried in the same conduit as the rain water.

Later, it became the practice, in many American towns, to provide sewers exclusively for the conveyance of sewage and industrial wastes and to construct an entirely independent system of conduits for the removal of storm water. This method of sewerage is known as the "separate system."

It was thought that the separation of sewage from rain water by means of the separate system of sewers would be complete protection to streams, as all the sewage could be conveyed to treatment works, even during times of storm, and also as the volume of sewage to be pumped or treated would be considerably reduced through exclusion of all rain water from the sewage conduits. In actual practice, however, these results are not obtained. Rain-water connections are made to the sewage conduit and sewage connections are made to the storm-water conduits, both willfully and through ignorance, and thus frustrate the anticipated purpose.

In many cases, a small town at first can only afford to install the sewage conduits of the separate system, and must tolerate the storm water flowing along the street surface until the municipal finances permit the construction of the supplemental storm-water conduits.

In towns having large drainage areas with relatively flat topography, the combined system is more likely to prove the cheaper, while in smaller towns, especially in those with rugged topography, traversed by streams, the separate system will probably be the least expensive as the storm-water conduits will be relatively short, because they can discharge to a near-by stream and thus will not attain such a great length and large size as if they drained the larger flatter areas.

For diverting the dry-weather flow of sewage and the first flush of storm run-off from a combined sewer system to an intercepting sewer, and for permitting overflow of excess storm water to the stream, many devices are used. Most of them include a dam across the invert of the combined sewer and a gate for the admission of sewage to the intercepting sewer, controlled by a float governed by the height of the water either in the intercepting sewer or in the combined sewer. Those devices with the fewest moving parts are most likely to be economical in both construction and maintenance and, what is more important, to be reliable in service.

The widest variation in rate of sewage flow will be obtained from short drainage areas having relatively steep territorial slopes and, conversely, the least range from large flat drainage areas. Also, small streams having pools or sluggish velocities need a higher degree of protection against storm-water overflow containing sewage than large streams with relatively high velocities. These factors must be borne in mind, in connection with intensity of storms, in determining the depth of flow in the combined sewer at which overflow may be permitted.

It is most important in designing sewers for the collection of either sewage or storm water to make careful estimates of the future development of the

area to be drained, in order to determine the ultimate run-off which the sewer will probably have to carry. Not only the average rate of flow should be estimated, but also the maximum rate, and, in addition, a factor of safety should be used to insure against future maximum flows which may cause the sewer to become surcharged. Minimum flows, especially during the period before complete development, should be considered, lest too low velocities occur and result in deposits and foul sewage.

The restrictions of a zoning code have a direct influence on the design and economy of a comprehensive plan of sewers. Industrial areas probably will produce the highest rate of sewage flow per acre. Closely built-up residential areas will produce higher rates of flow per acre than the open development of suburban residences. In a town built on a sloping site, on the bank of a stream, it is probable that zoning regulations will confine the industries near the banks of the stream, and the open residential area will be at or near the high land. These conditions make possible the use of sewers of relatively small capacity at the summit ends and do not require provision for the highest flows per acre until the lower ends of the sewers are reached.

Fresh sewage has practically no odor, but in stale sewage the decomposition of the organic content is the cause of offensive odors. Therefore, the sewer system should be designed, constructed, and maintained so that the sewage will be delivered to the place of disposal or the treatment works in the shortest practicable time and at such velocities as will prevent deposition of solids. Therefore, sewer inverts should be laid on straight gradients, junctions should be made so as to prevent the formation of eddies, minimum velocities should not be below 2 ft. per sec., and the interior surfaces should be made as smooth as possible.

Thorough ventilation of a sewer system is also needed to help retard the decomposition of the sewage and to reduce the hazard to workmen who may enter the manholes or the sewers. At least on new sewer systems, house sewers should be directly connected without the use of the "main house trap", and ventilation should be provided through the main house stack of the plumbing system. Such ventilation makes unnecessary open-top ventilating manhole covers which, when located in other than most durable street pavements, afford opportunity for grit, cinders, and other detritus to enter the sewers from the highway and, in the absence of high velocities, cause deposits in the sewers or, if carried forward, cause trouble in the sedimentation tanks at the treatment works.

In all cases where sewage must be pumped or treated, or where such may be required in the future, the infiltration of ground-water to the sewers should be reduced to an economic minimum through the use of tight joints in pipe sewers and impervious construction in masonry sewers.

Since industry is the life of many communities, it should be encouraged, and not hindered by restrictive legislation prohibiting the use of sewers for the conveyance of suitable industrial wastes. The treatment of industrial wastes at the point of origin is costly, oftentimes prohibitively so, unless by-products can be recovered. It is to the interest of a community to receive suitable industrial wastes into the public sewer system, except those which

may injure the structure of the sewers, interfere with the free flow of the sewage, seriously affect the successful operation of sewage treatment processes, or cause tastes and odors in public water supplies taken from the stream receiving the sewage.

In cases where industrial wastes are accumulated in tanks in the factory, the peak load on the sewer system or the possible difficulty in operating the sewage treatment works can be reduced materially by the discharge of the industrial waste over a relatively long time rather than by intermittent discharge in large quantities.

Sewer systems, like any other utility, require maintenance. Because the sewers are unseen and underground, they are usually neglected. If deposits form, they should be removed, and damage to the structure of the sewers should be promptly repaired. How else can the sewers render efficient service?

The main sewers of a town are almost always laid in the valleys of the drainage areas and, therefore, convey the sewage to some near-by watercourse. This means that the water-borne filth of the town is transferred from the premises to some body of water.

The conservation of natural resources in the surface waters of the United States demands that streams and other bodies of water must not be rendered unfit for normal use by the undue discharge of sewage into them, and several States have enacted laws to control the matter.

The protection of the public health requires that streams used as sources of public water supplies shall be maintained in such a condition that the water can be purified safely, at a reasonable cost, and made fit for drinking purposes. Common decency calls for clean streams, that is, free from nuisance to sight and smell of sewage origin. Therefore, the degree of treatment to which sewage should be subjected, in order to meet these requirements, varies between wide limits, depending on the use and conditions of the receiving body of water.

Disposal of sewage by dilution is a proper scientific method in cases where there is an ample flow of water to assimilate the sewage inoffensively and maintain the stream in a clean condition, and where the discharge of sewage will not endanger a source of public water supply, shellfish beds, bathing beaches, or other uses requiring a hygienic standard.

Where the receiving body of water cannot assimilate the crude sewage, but can receive the dissolved and non-settleable solids without danger of nuisance or menace to the public health, the required degree of treatment can be obtained by sedimentation of the sewage to remove solids of appreciable sizes, or even, in a few cases, by fine screening.

The separation of the deposited solids or sludge from the liquid part of the sewage, as in the Imhoff tank, or where the sludge is removed to a separate sludge digestion tank, materially tends to prevent the increase in the oxygen demand of the tank effluent, which occurs when the decomposing sludge is in direct contact with the settling sewage. In small installations, however, where efficient operation is not likely to be obtained, the much abused septic tank often proves far more successful than many idealists believe. The real

difficulty in the sedimentation of sewage is in the de-watering and final disposal of the resulting sludge, and not in securing an effluent free of settleable matter.

Undue economy in design in order to reduce cost of construction frequently results in the sludge compartment of sedimentation tanks being too small to permit retention of sludge long enough for thorough digestion of the organic matter; and during the operation of the tank the careless withdrawal of the sludge places on the drying bed a more watery sludge than need be and one containing undigested organic matter. These are the causes of the offensive odors during the slow drying of the sludge. With a tank of proper design and with skillful operation, the danger of nuisance from drying sludge on sand beds is reduced to a minimum.

In cases where the receiving body of water cannot assimilate the putrescible dissolved and non-settleable matter in the effluent of a sedimentation tank, further treatment is required to reduce the oxygen demand. This may be obtained through oxidation by means of the trickling filter, contact bed, or intermittent sand filter, as may be deemed most economical or suitable to local conditions.

Where conditions, distance, and time of travel between the point of discharge of partly or completely treated sewage and the intake of a public water supply are insufficient to insure an appreciable death rate of the pathogenic organisms in the sewage and produce at the water-works intake a raw water susceptible of economical and safe purification, the disinfection of the sewage effluent should be adopted as an additional safeguard.

Both chlorinated lime and chlorine gas are used for this purpose, the quantity depending on the organic content of the sewage effluent and the degree of removal of bacteria required. A solution of chlorinated lime may prove more reliable than chlorine gas, because of the slower action of the former in permitting an opportunity for penetration of solid particles.

The addition of a chemical germicide to crude sewage naturally can only affect the bacteria present in the liquid in small particles and on the outside of large particles. When, later, the large particles disintegrate, they liberate the organisms which have been unaffected by the germicide, and the fancied protection is nullified.

In their design and operation, sewage treatment works involve many scientific principles. Successful and economical results cannot be obtained except through skillful design and intelligent operation. A sewage treatment works can no more run itself than a power plant, yet many municipal officials think it can, and act accordingly.

In conclusion: Every municipality should have a carefully prepared comprehensive plan of sewerage and construct the various sewers thereof as necessity arises and funds can be made available.

The sewers should be designed, constructed, and maintained so that they will promptly convey the sewage in as fresh a condition as possible to a proper place of disposal.

The degree of treatment of the sewage should be sufficient to maintain at all times the receiving body of water in a clean condition and fit for normal reasonable use.

HOUSING LEGISLATION AND FINANCING

By Allan Robinson,* Esq.

The housing problem is not the problem of the rich or the moderately rich, so much as it is that of the poor or the moderately poor. In New York City there are numerous vacancies in the houses occupied by the well-to-do, but practically no vacancies for the poor. Such vacancies as do exist, have been created, in part, by new construction of better class apartment houses and, in part, by a migration from the city into the outlying districts. The poor, however, cannot migrate; they must live near their work. The rents of the poor have risen quite as much, proportionately, as the rents of the rich, and they have met this increase either by doubling up wherever possible, or by moving into poorer and less sanitary quarters. There were a great many vacancies in former days in the cold-water flats, but these flats are now filled with families who formerly lived in steam-heated apartments. The menace is great where a large part of any community is compelled, by economic conditions, to lower its standard of living. No true solution of the housing problem will be reached until ways are found to furnish homes to people of moderate means.

Statistics indicate that a great deal of building is being done in New York City and vicinity. The Building Trade Employers Association reports the following comparison for the period from January 1st to December 10th, in the years 1920 and 1921:

	1920	1921.
Manhattan	$96 686 325	$121 862 741
Bronx	18 130 600	65 043 648
Brooklyn	55 985 373	90 341 510
Queens	32 701 277	72 984 340
Richmond	7 990 485	3 551 550

This construction will provide homes for 21 730 families, and predictions are made that a greater amount of construction will be undertaken in 1922 than in 1921. Although these figures are encouraging at first glance, they are not convincing. On the contrary, an examination of the causes which are responsible for the present building, indicates that there are certain fundamental defects in the situation, which are anything but encouraging. In the first place, this spurt serves to keep up the wages of the building tradesmen at a time when wages in all other trades are receding. In the second place, the types of buildings that are going up do not offer any real help to the people who most need homes. At present building costs, accommodations cannot be furnished at less than $15 per room per month, and this is more than the average skilled workman can afford to pay.

Furthermore, not only are the single-family and the two-family houses that are being built on the outskirts, erected only for sale and not for renting, but the apartment houses that are going up in various parts of the city are

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being erected by builders for the sole purpose of sale. Investors are not helping. On the one hand, there is an artificial stimulation of speculative building growing out of the effect of the tax exemption ordinance, and on the other, there is the retarding effect of the rent laws. As time goes on, and the period of tax exemption grows shorter, the effect of the tax exemption ordinance will weaken. As long as the rent laws are in effect, investment building of multi-family houses will be practically nil. Therefore, the appearance of prosperity in the building trades should not deceive, but should direct, at the earliest possible moment, thoughts and energies toward modification of the rent laws, so that tenants may be protected and investment funds attracted to the building market.

Mortgage money is more plentiful than it was in 1921. The Metropolitan Life Insurance Company states that from January 1st, 1920, to December 16th, 1921, it has loaned $18 774 357 on 5 068 dwellings and $49 205 750 on 589 apartment houses, or a total of $68 080 107. Of this large amount of mortgage money, which has been made available for housing purposes, $45 680 200 has been placed in Greater New York. Although these and other institutions are making generous appropriations for housing, private funds which in ordinary times would seek the mortgage market, have been withdrawn from it. The reason for this is not far to seek. Mr. Clarence H. Kelsey, President of the Title Guarantee and Trust Company, states that:

"The present high surtaxes make 6% mortgages actually produce very little more than 3%, and so they are much less desirable than other securities. Farm loan bonds are exempt from the income tax. There are more than $20 000 000 000 of exempt securities in this country already. * * * Men who are in the business of attracting money into the mortgage market see very plainly that there are only two practical remedies, one is to reduce surtaxes, so that money will not be sucked into the tax exempt securities, and the other is to exempt a given amount of mortgages * * *."

The exemption suggested by Mr. Kelsey is the same as that recommended by Senator Calder's Committee on Reconstruction and Production. This Committee reported in favor of an amendment to the Revenue Act of 1918 to exempt from taxation interest on loans up to $40 000 on improved real estate used for dwelling purposes, when such loans are held by an individual. If there is going to be any sucking of funds into tax exemption securities, let some way be found to suck them into building mortgages. Tax exemption is not defended, but as long as it is recognized by the law, it should be extended so as to provide protection where protection is most needed. As the law stands, the effect of tax exemption provisions works in the interest of the rich. The exemption suggested would tend to help the poor, because it would attract capital into the moderately priced houses.

The true solution of the housing problem, however, is not to be found in legislation or in making available a greater amount of mortgage money. The true solution is in the production of low priced homes, and these cannot be produced with labor costs at their present levels. At the cost peak, efficiency was not over 50% of the normal. Bricklayers who in the pre-war days were accustomed to lay from 1 500 to 2 000 bricks per day, in 1920 were laying only

600 bricks per day. In one case, with which the speaker is familiar, a man laid as few as 450 bricks per day, and, when the contractor discharged him for inefficiency, the whole gang of bricklayers walked out. What was true of bricklayers was also true of other building trades. The wages of building tradesmen increased, on the average, about 100%, and their efficiency decreased approximately 50 per cent. Building costs are still 100% higher than prewar levels, and until there is a further improvement in efficiency and a substantial reduction in wages, it will be impossible to produce homes for people of moderate means, to rent on the basis of $10 per room per month, which is all that these people can afford to pay.

The effect of improper housing is felt not only in the physical well-being of a community, but also in its civic and moral aspect. Unless something is done in the near future to house comfortably and in a sanitary way, the large mass of the people, the citizenship of the country will suffer. It is of the utmost importance, therefore, that waste in building be ruthlessly eliminated, and that building tradesmen be induced to accept in their own interest a lower wage than they have been receiving.

ZONING LEGISLATION AND FINANCING

By Edward M. Bassett,* Esq.

The speaker has given especial attention to the subject of zoning, and he will try, in this paper, to cover a somewhat broader field than legislation regarding zoning. In the suburbs of New York City, and in most of the large cities, proper restrictions have always been applied to high-class residences. It is only since zoning has come to the City of New York that the small house has had any assistance or protection, such as the private restrictions afforded to wealthier people; but private restrictions, even if they were adequate, were troublesome because they often resulted in litigation. They were usually fixed for a term of years and thus would not permanently affect a neighborhood after the restrictions had lapsed. Non-conforming buildings, apartment houses, and business would invade the locality where the restrictions were contradictory in character. The Courts were apt to set them aside because of a change of character approaching the neighborhood, and there was always need of celerity on the part of the private citizen in order to protect himself under these private restrictions, because if he was guilty of negligence, the Court would consider that his rights had evacuated.

Zoning has brought better protection to the homes of the well-to-do than private restrictions; but a remarkable development is perceptible in that it is beginning to protect the homes of people of moderate means. The "E" zones are residence districts, in which is permitted the construction of a building covering not more than 30% of the land. The intention was to protect good suburban development; but during the last three years (1918-21), there have been at least eight petitions to place the people of moderate means in "E" zones, where apartment houses and block houses are barred.

One of these localities, known as Glenmorris, in the Borough of Queens, has buildings valued at from $6 000 to $8 000, many of them, however, about $6 500. Those small homes are protected, and the person of small means who has bought one, can be sure of having a home for himself and his children, thus bringing about a permanency and a desirability of small homes in Greater New York. It is greatly to be desired that even in New York, the workingmen should have his own home.

A home owner is a good citizen because he takes an interest in the community, the streets, the upkeep, and the expenditure of money by the city. In owning his own home, he is in all ways a better and a thriftier citizen than one who is paying rent. It is difficult to accomplish much in Greater New York along this line, and yet the rapid transit and the zoning are bringing about the possibility of a person of very moderate means owning his own home.

The old City of New York was a long city, with a small area of land and long distances of travel. Development during the last 12 years has tended

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to produce a round city, which has the greatest area for the shortest distance from the center.

Mr. Robinson has covered the fundamental considerations regarding multi-family houses and the reasons why they are not being built in New York City. Zoning has an application to multi-family houses. It does not allow the congested building of residences that could once be placed in New York City. At present, there are the "E" zones, and also the "D" zones, or about 65% zones, suitable for block houses and small apartment houses, in which is prevented the erection of unnecessary multi-family houses. There are enormous areas of "D" districts in Staten Island, Kings, and Queens.

The zoning of New York City, however, should have a more intimate relation to the prevention of congestion than was considered at the time the zoning was planned and passed by the city. It was a novel enterprise. To what extent the Courts would uphold it was unknown. It was desirable to go as far as there was certainty that the Courts would back it; and when that was discovered, the intention was, in some respects, to find methods of more adequately preventing congestion.

One method is certainly the introduction of districts in which there will be a greater percentage than in the "E" district of 30%, and less than the "D" district of 60%, something like a 47 or 50% district.

New York City is slowly becoming a better place for workingmen to have their homes. The suburbs are being opened by rapid transit as they never were before; and the increase of the periphery of Greater New York and the accessibility of this property by transit have been so great that the land available for this purpose is comparatively inexpensive.

This brings up the subject of police power. All the regulation that can be brought to the large cities, in behalf of better protection of families in their homes, must be done mainly under the police power. The police power is that power of the State, which can be invoked for the protection of the health, safety, morals, and general welfare of the community. Zoning is perhaps as generous a grant of that power as has ever been made by States to cities; and the Courts have affirmed it in the City of New York, the highest Court of the State having declared that zoning is a constitutional invocation of the police powers of the State. In all these invocations of the police power for the protection of housing, are some fundamentals that have come out clearly in the litigation that has arisen in regard to zoning. The Zoning Enabling Act, or rather the amendment to the Charter enabling the City of New York to pass a zoning resolution or ordinance, was intended to donate all the powers relating to height, area, and use of buildings.

That effort was not successful. A part of such powers was granted, but if one wishes to include in zoning a limitation of families per acre, it is desirable to state in the Enabling Act that the city shall have the power to restrict by families per acre; or if it is to be done by any other method there must be a statement that is sufficiently adequate to allow the city to exercise that particular method of establishing the police power. The speaker wishes to urge any one who endeavors to draw an Enabling Act that will grant to

cities the power to protect buildings, to take great pains in the language used, preferably using broad language, that will allow the largest scope of the power. If that cannot be done, then enough should be included in the way of specification to accomplish the result desired.

Some of the zoning laws of the State are not being upheld by legal decisions because the Enabling Acts were not adequately drawn to accomplish the zoning which the cities placed in the ordinance. In the City of New York there is a trend of sentiment that affects this question. Mr. Robinson referred to it plainly. The speaker is not sure that he can be more explicit than Mr. Robinson, but he wishes to suggest that there is a sort of social insistence—he will not call it Karl Marx socialism, but along the line of this great wave that is going on all over the world—the voter considers that by his vote he can affect the price of the things he consumes; and from that he concludes that he will by his vote affect the price to get them as cheaply as possible. That reflection of his vote operates against public utilities.

An honest public utility is as desirable for a city as an honest apartment house. It is an honest investment in something that the people need, until the city can construct and operate the same thing for them. Most of the cities, in the United States at least, have not arrived at the point where they can do that; and so the voter casts his vote in a way that he thinks is going to lessen to him the price of the thing consumed. This relates to transit and other utilities. It relates to apartment houses, as is reflected in these laws that seek to make the consumer the favored one.

It will be noticed that all these laws affecting prices to the consumer are "nailed down" in the City of New York and cannot get away. They are not, for example, like wheat, because wheat would be sent to Philadelphia, or some other place; but when a thing is actually "nailed down" in New York, there seems to be an advancing sentiment that the consumer by his vote will make the price of that which he consumes.

One of the dangerous elements in legislation along this line is that it prevents capital from coming in to build what is necessary for the consumer. Sooner or later, this delusion will end; when the voter perceives that his apartment house, his utilities, and the things he needs, are not being afforded to him, he may then realize that one of the reasons, at least, why they are not being afforded is because by his vote he has driven to other places the capital that would increase public utilities and apartment houses.

An investment must earn a fair return and a fair return is that rate which will induce other money to come for additions or duplications of that same thing. If legislation brought about by the consumer's vote makes it impossible for a fair return on new investment, then that money is going into something else. The consumer will discover that there must be a change of attitude in order to attract capital, which is absolutely necessary for the city.

One might say, the city itself can do all of these things; but the experience of Vienna was most disastrous; and in New York City the speaker presumes that until tenure of office is more fixed, until there is greater fidelity to public duties, until there is an opportunity for the public administrator to obtain

a greater skill in the performance of his work, it is going to be doubtful whether the city can substitute itself in apartment houses or small homes, or in many of these other things, for the private investor; and if one can conceive that for some time at least the private investor must be depended on, then, probably, among other things, there will be a change of attitude that will attract capital to the great cities.

LEGISLATION AND FINANCING FOR HOMES

By William H. Ham,* M. Am. Soc. C. E.

The whole housing problem is replete with disorderliness. The financing is left at a critical point by the large institutions, this being known as the point of double security, where it is left to be finished by those who drive sharp bargains. Construction methods have little in them that should not be changed. The styles of houses that have been most largely produced, do not have the approval of architects or engineers. The people are still struggling to throw off the yoke of mid-Victorian badness in the East and what may be well styled the "near-Pullman" in the West.

During the World War and the period of activity which immediately followed the armistice, a tremendous movement of peoples took place, which, in general, was a gathering around industry. These people have usually been below middle age, for it is always youth that is mobile. They have been and are still restless—as youth always is restless—they have added a streak of independent action to the normal restlessness, because so many of them have been in the Army where home restraints were removed for the first time. They are also more independent than is usual among Americans, because of the invigoration of physical development of these able-bodied young men.

The training of the Army, and of those who stayed at home as well, has been such that there is no place left for the unsanitary, ill-lighted, badly designed, uncomfortable tenements, and the speaker is sure that in the immediate future there will be found an entirely new clientele for the new homes. This clientele will present new demands which must be recognized in the building program. The dull, dismal, colorless home which has been so profusely built in the last fifteen or twenty years must be eliminated.

Every radical change develops confusion at the start. A confused state of mind is evident, regarding the building of homes, and the few clear, outstanding facts should be emphasized so that they will have the clarifying action of public discussion and, when developed, have the endorsement of public opinion.

The speaker will present a few statements from a leading thinker of our generation, who has given attention to the fundamentals with a clear vision, written in the mature years after an extremely busy and thoughtful life. Dr. William J. Tucker says:

"The social curse of industrialism as it now exists lies in its effect upon the disposition and temper of industrial workers. It has taken away from them the zest for work, than which nothing is more necessary to social progress. This alienation in spirit of the man from his work is as evident in the higher ranks of industrial labor as in the lower ranks.

"Industrialism has put him under the domination of the machine; it has subjected him to various conditions not of his own choosing; and it has deprived him of the stimulus and incentive to private ownership.

"No man can be satisfied with his work, who is not allowed a share in the responsibilities and rewards of private ownership. Industrialism, under present conditions, deprives its workers of this satisfaction. It makes no provision for

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their individuality. It swallows up the individual in the class, leaving him in just complaint over his unsatisfying lot. And the most disheartening fact is that those who suffer most from this lack in industrialism have sought for compensating equivalents rather than for a reform of the system."

These statements stand out because of their clear expression of the cause of the difficulty, which is industrialism working its change on the community and the suffering of the individual as a result of service to industrialism.

The industries themselves are changing, and great strides have been made in some important organizations toward the distribution of ownership. The speaker refers especially to the United States Steel Company and many other companies, which have largely distributed their stock to satisfied employees on a basis of purchase within the power of the wage earner, and by this distribution have earned the good will of a large number of their best workmen.

The building trades have been sufficiently exposed so that the people know there have been frauds by the cheap, speculative builder, excessive interest rates charged by the money lender, and a very low return has been given by labor, which has shirked. The industry has been hampered by bad laws, both written and unwritten, and by unfortunate customs and compromises by the builder through his associations and the workman through his trade unions. These influences have been extremely subtle and far-reaching.

Indications, however, in the buildings now under construction, show that more work is being done by the trained mechanic, because of the keen competition for the job and the speaker believes also because the mechanic himself has learned, in part at least, that limiting his output is un-American and not in accordance with the rules of the game. However, a rational apprentice system is not being developed. Interest rates have been excessively high, but are now lowering. There is little difference between the statement of the financial leader when he says "Money must receive for interest what it is worth in the open market", and the statement of the labor union delegate when he says for his men in the union, "We can get the high wage and do a small day's work and we are going to do it". The wages of money and the wages of men constitute the principal items of cost in the building and owning of homes. Every evidence is seen of a return to normalcy.

In 1911, the relation between the material cost on the job and the labor cost for installation was in the proportion of approximately of 70% for material and 30% for labor. In 1919, this proportion was changed to 40% for materials and 60% for labor, and, at present (1921), it is about 50% for each. The cost of materials on the job is largely a matter of labor in itself.

The Engineering Profession should take up and study sufficiently at this time the whole problem in such a manner as to furnish to the builders of the great industrial period ahead, a clear-cut outline of available materials, efficient methods of construction, and a rational means of financing. The use of labor effectively is the one great problem for which engineers are qualified best to solve.

Let the industrial and financial groups to which the housing problem is extremely important, be assured that every phase of it has been studied with

an analytical care so thorough as to eliminate all the excesses which have been so marked that the public has taken cognizance of them.

Let the engineering societies join hands with the architectural institutes, the mechanical societies, the banking group, and the manufacturers' associations in a joint effort to lay down a clean-cut specification for the materials which should enter into the construction of the homes of the people, the proper methods of building, and the proper costs for various items of construction, going sufficiently deep into the problem to reach the proper output for a day's wage in the various trades, and, in addition, establishing a proper plan of finance which will insure a wider distribution of ownership.

A committee, working with broad views on this problem, will find a tremendous amount of variety in the requirements of homes and the regular human factor which enters into all the problems and, as well, a definite prejudice in regard to details and methods. All these are such that careful analysis will lead to a clear view of the proper program for building in the immediate future 1 000 000 houses in the United States.

The speaker wishes to point out a few of the fundamental items which must be reviewed, changed where found faulty, and standardized after study. These will be classified as follows: Construction items, family needs, and financial methods.

Construction Items.—Of the items of construction which will show the greatest result in economy and the greatest change in methods of building, the following seem to be the most important.

Excavation methods are wrong because they employ either hand labor or horse labor in the majority of cases. Small machines are needed and must be developed for the proper handling of small volumes of earth. Large machines for heavy excavation have been developed to the highest point of efficiency, but little effort has been made with the small machine.

Foundations themselves have been built by "rules of thumb", which have found their way into many building laws in various cities of the United States and have affected seriously the cost of construction. Foundations should be factory made, transported and erected with proper machinery as a part of the manufacturing plant equipment.

Timber framing has been used for practically all the development since the Civil War, by "rules of thumb" and not in accordance with engineering principles. Joists and studs have been placed 16 in. on centers, principally for the reason that the waste lumber which is made into wood lath, will not span any greater distance with safety; whether the span is large or small, the normal spacing is 16 in. Although some variation in the depth of the joist has been made, this variation is not in accordance with the load requirements.

Framing methods are fairly well established, but the cutting of timber in the forest still follows largely the 13-ft. rule. Tremendous waste is caused by the cut-off ends of lumber. The whole lumber industry and the use of the lumber in buildings should be standardized and put on an engineering basis.

The next item of importance is piping, which includes the heating and plumbing throughout the building. Little work has been undertaken for

standardization in the plumbing industry, and that little has been resisted by the plumbing trades at every turn. Sanitation of cities outside of the buildings has been given intimate study by expert professional men, and the Society has furnished notable examples who have done a wonderful work, whereas practically no progress has been made toward the solution of the problem of plumbing or inside sanitation.

Hundreds of cities have as many different building codes, intricately drawn, authorized by a common council composed of mediocre laymen, influenced by a group of badly trained mechanics who have insisted on excesses without question, under the guise of sanitation, and not a doctor or a sanitary engineer, with a few exceptions on Government work, has ever been at a hearing or had a word to say about these building laws. Regulations in the trades themselves have limited output to an excessive degree.

The speaker will cite one example that is standard throughout many of the States. In the piping trade, no pipe 2½ in. in diameter or less may be cut in the shop by machine, but must be cut by hand on the job. Nine-tenths of the pipes in a building are probably under this diameter, therefore nine-tenths of the pipe for plumbing and heating work is still cut by man power on a temporary bench, with hand tools. All this can be overcome by standardization of the plumbing and heating layouts so that the full requirements can be shipped to the job by the factory, ready cut for installation and largely fabricated.

This item of piping alone demands the most careful study of some competent engineer who will have the courage and convictions to go into this problem with the knowledge that forceful methods will be required, since the opposition of the plumbing industry, not only of tradesmen, but of manufacturers as well, will be encountered at every turn and will continue to oppose until the strength of public opinion has given endorsement to a revision in this part of the building process.

The speaker believes the elimination of back-venting and the substitution of anti-siphon traps, with relief venting, or loop venting (which is used by the Government) should immediately be endorsed by the engineering societies since sufficient data are available to warrant this conclusion.

Plastering by the wet process in average buildings constitutes a setback by bringing water into the building at a time when everything needs to be dry, wetting the timber and trim members, and causing a delay and serious damage to the plaster itself on account of the swelling of the materials to which it is applied. Plastering by the wet process is wrong, and the dry finished product substituted therefor is most urgently required.

The objection to the wetness of plaster is only one minor objection. The more important objection to plaster as now applied is its continuing destruction in the house itself, which constitutes one of the most serious items of depreciation.

There are substitutes on the market which are being largely used, but they have not been developed sufficiently, either as to quality or as to size of sheet. Co-operation between the manufacturers of plaster substitutions and the archi-

tects is necessary to develop to full value the substitute materials which can be used for the inside walls of buildings. The speaker has a suggestion to offer to the architects in this particular, that is, to divide as far as possible the inside surfaces of the rooms into panels with the use of door and window trim extending from floor to ceiling and to return to the extensive use of a proper wainscot, simple in form and developed to work satisfactorily with the plaster substitute.

By this method a large part of the house areas would be reduced to small-sized units, easily transported and simply installed. The ideas of our ancestors in the exterior form of the building are returning, and if these important details are emphasized on the interior, a large number of the plaster difficulties will be eliminated.

Fabrication of house parts in the shop in the locality of the building site is the next important item. By this is meant such items as door frames and trim complete, window frames in like manner, and all dormers complete to the last detail, including the window, with the hardware installed. These fabricated parts of buildings, to be made by standard shop drawings, transported in the fabricated state, and installed by men trained to this kind of work, will lower the cost of construction and, at the same time, give the architect an opportunity to express himself, because the work is done under mill conditions and in the locality where the house is to be built.

In the construction of small buildings there are hundreds of items which will lend themselves to the same analysis as the few mentioned.

Family Needs.—The needs of the family can be discussed at unlimited length and still leave unsaid much of importance. The fundamental principles of demand and the reasons therefor as they apply to the home of the workingman, classified by the industries as a "wage earner", and the small salaried man who is a step only above the wage earner in independence of action, can be codified quite simply and the speaker has undertaken to set down those items which emphasize themselves most clearly, as follows:

1.—The workingman demands a decent home.

2.—The minimum requirement for one family is entirely different from the minimum requirement of another family in the number of rooms. The minimum demand changes during early married life with the advent of children.

3.—The workingman will seek as near as possible a minimum which satisfies his requirements.

4.—Three rooms and bath constitute a minimum and make an exceedingly popular unit. Next in popularity is four rooms and bath; next, five rooms and bath; after that, six rooms and bath in an individual house.

5.—The workingman demands a share in ownership. This demand is not fully crystallized, but should be stimulated as a great stabilizing influence in manufacturing communities.

6.—He will undertake a contract which appears to be free from paternalistic influence, if the start in the form of payment is small. This is proved in the housing work in Bridgeport, Conn.

7.—It is not practical to build a three-room individual unit suitable for fee simple ownership. The same is true of a four-room unit. The five-room unit is the smallest individual house that should be built. The six-room house is the best isolated individual unit.

8.—The workingman must pay the price that is necessary for a decent home, and the wage must be sufficient to allow the payment for a decent, economical home.

9.—The large amount of money necessary to support a home-building program demands that the funds shall come from the people who work, and in small units.

10.—The money will be obtained more cheaply directly from the people than from any other source. The demands from the people for use of their money are two: Safety of investment and liquid character of investment.

11.—Cities grow in proportion to the number of families that are satisfied and not in proportion to the birth rate. Rapidly growing cities attract young people, because it is youth which is mobile; therefore, it is incumbent on these cities to provide homes for young people and that is what Bridgeport has done.

12.—Unattractive home units will interest only those with sharp practice and speculative plans in mind. The attractive home interests both those who are in the employer class and those who are in the employee class. The many visitors to the Bridgeport project is evidence of this influence. Simplicity and good taste are fast taking the place of extraneous frills. The growth of this new style will be made clear to the worker by a slower process than to the employer, but will surely find its way into the popular mind during the next few years.

13.—A plan of graduation from the minimum three-room unit to a permanent fee simple ownership of the six-room house is desirable, and Bridgeport has built its houses with this in view.

14.—Economy demands the three-room home as a temporary dwelling only. The four-room unit should follow the three-room home with the advent of children, and the five-room unit should follow the four-room house.

If there were no houses and no customs to contend with in the building trade, the problem would have fewer complications for the building of the various types of homes that are required to meet the family needs could proceed. This is not the case and, therefore, what is here must be used as far as possible and additions built thereto, which will, when completed, make the whole structure a harmonious product. There are bad tenements and many of them; these must be rebuilt. There are many over-sized, ugly houses which, of necessity, must have something done to better their value and improve their appearance.

As the speaker sees the program of addition, it divides itself into three distinct items of construction, each of which is important.

The first item is the organization for and the building of a large number of small unit homes, in groups over industrial cities wherever vacant spaces are available. These groups are to take the form of a village within a city, each community having its common playground and private gardens. These

small homes are to be variable in size and to be so small in number of rooms as to reach the minimum requirement of small families of the working classes, but, at the same time, furnish proper surroundings for ideal living conditions with the home. These homes must be in part owned by the people who live in them, and the investment which represents their true value should be divided so that it can be readily distributed to these people who begin living in the city village and owning some small share, that share growing week by week.

The speaker would go so far as to make the shares of ownership extended to the beginners in family responsibility and home purchase a liquid investment so that it can be exchanged readily for a larger and more pretentious home of the same or other development, or for a first investment in an individual home.

The second form of building clearly demanded by the people is the small individual house, averaging six rooms and bath to each home, built on its own lot. These small, well-built houses should be constructed from carefully prepared plans and should be sold on an installment basis to those who have a sufficient amount of money to make an initial investment, equal at least to the risk in the property. The organization of a building program for these homes should carry with it the opportunity of exchanging part ownership in the "city village" previously described, for the first investment in a house which is to become the property in fee simple of the occupant.

The third item, which is no less important than the other two, is the rebuilding of run-down tenements in the slums and the raising of these to a standard which will qualify as an American home for the lowest wage earners. Some of these structures are so bad that modernization is impossible, and they should be eliminated. Most of them, however, will yield to a radical re-building process and give results which for a temporary expedient, at least, will suffice and save a tremendous amount of construction cost.

A real program of rebuilding to bring these inferior homes up to standard should be outlined, and standardization of all such items as are possible should be introduced.

The problem of multiple-family homes on expensive land in large cities is another function of the housing problem, which needs study and which should incorporate the distribution of ownership to the individual and the introduction of community amenities commensurate with the class for which this type of housing is built.

Financial Methods.—The speaker has confined this paper, principally, to three forms of homes. Although this is not all the program, it is a definite and substantial part, and leads directly to the necessary phase of the situation, namely, the financing of homes.

The city village is the most idealistic and probably the hardest to finance from two standpoints, of any of the types recommended. The first of these is lack of knowledge on the part of the people who control and direct the money; and, second, because of the necessity, in order to secure successful results, of bringing into this financing the small savings of many people.

It will be necessary, for a start to be made in building up of these "city villages", for wealthy individuals and corporations of high standing to under-

write the initial expenditure, distributing the ownership at a later date to the individuals. A large apartment-house movement has already started in some of the largest cities along this line. Co-operative ownership is not recognized as much in America as in England, but still there is the marked example of the building and loan associations or co-operative banks, as they are called in some of the States. These are increasing at the rate of $500 000 000 per year in assets and are simply co-operative institutions founded on the integrity and confidence in members and borrowers.

From experience with the management of four of these city villages, the speaker is convinced that harmony of interests can be readily developed, and a supporting influence of pride will spread with wholesome contagion.

The next problem is the financing of the individual home which some time is to be owned in fee simple by the more prosperous, further advanced, thrifty worker. This has less call to the imagination, and can be simply stated as to its source of capital and the method of accumulation of ownership by the purchaser. To every family of good repute in every industrial city, there should be offered an opportunity to buy such a home when this family has, by thrifty saving or otherwise, accumulated $1 000. This amount, if the home is built on a proper basis and of durable materials, will represent all the risk of loss that such buildings in the average should show.

Association of capital in a program that will produce these homes and protect them with proper and definite surroundings and wholesome restrictions, can bring better results to that community than any other form of investment that the group of financial men can support.

The definite, clean-cut demand on the part of the people for a method of buying is simple, easy to follow, and good financing, is understood by the people, and is already in practice in many places. This definite, concrete demand of the people is for a form of deferred payments on home property in a single monthly installment, with definite amounts established. The use of a first, second, and, sometimes, a third mortgage, each with their hazards and complications, and the vicious bonus system now thoroughly established, are disliked by the people, who wish to buy at a fair price and who can buy and pay for the house, with proper interest rates, when it is built at a fair cost value.

This great problem of furnishing the money above the initial payment of the purchaser, which averages much less than half the cost, is one which must be woven into the banking and other institutional finance, in such a manner as to use the great amount of money available through the savings institutions and the National banks, insurance companies, building and loan associations, and the private lenders.

The tremendous amount of money available for first mortgages on one-half of the value is evident, $400 000 000 being available through the National banks for one-year loans on real estate direct. Savings banks and trust companies have an equal or greater amount available in the same way, but for a longer period. The building and loan associations are increasing assets, available for long-time loans, at the rate of $500 000 000 per year. This last agency

is the only one that can legally advance very far beyond the double security point.

Mr. Walter Stabler, of the Metropolitan Life Insurance Company, has outlined the possibility of a close co-operation of the already established building and loan associations and the insurance companies, and the speaker wishes to quote briefly from a paper by him, presented to the Chamber of Commerce of the United States, in January, 1921, as follows:

"This, therefore, brings us back to the proposition of first mortgages of 50% being made by insurance companies, savings banks, and other lenders through, or in co-operation with, the Building and Loan Association, with the associations making the additional loan up to a total of, say, 80%. I am not aware that such a plan has ever been put into effect, certainly not to any great extent, but I feel very sure that it is safe for all the parties interested and that it should provide large sums of money, not now readily available. Such funds could be secured in the briefest possible time and without the red tape apparently inseparable from Governmental agencies. I hope the associations will give it serious consideration. I believe it is a question, the solution of which, is largely in their hands. They can by this, or similar means, provide the second mortgage money, so much needed and so difficult to obtain, to which I referred in the early part of this paper."

The financing of the rebuilding of run-down tenements is a private matter that can be largely influenced and stimulated by co-operation of the loan institutions, and the speaker suggests this as a means toward the development of a wholesome beginning in rebuilding.

Let the lending institutions in the industrial cities publicly offer to increase the mortgage loans on run-down tenements to the amount of added cost of improvements, if these improvements are made in accordance with a standardized rebuilding method, outlined by some substantial organization which shall, by its merits, have public endorsement.

Let the industrial cities, through their governing bodies, exert the full police power to eliminate the bad character of buildings and urge their improvement by force of this police power. In many cities, there are buildings that are so far below the standard that the power of the city to close them could not be questioned. This power should be exercised for the betterment of the condition of living, and should be done at once.

Let a committee from these associations which have been mentioned, and such others as should be taken into the problem, formulate a clean-cut program, endorsed and approved by all, and let this program be publicly distributed through the various channels available, such as building supply houses, banking institutions, and the architectural and engineering society journals, and a result of harmony in the whole problem may be expected.

LEGISLATION AND FINANCING

By John Ihlder,* Esq.

Three statements belong at the beginning of any discussion of these two allied subjects, for they are allied in fact as well as by a conjunction.

First.—Before the World War any mention of the housing problem conjured visions of slums, whereas, to-day it conjures visions of the comparatively well-to-do, including United States Senators, fighting against increased rentals. Before the war, we had to remind our hearers constantly that housing is a community problem affecting even the well-to-do. To-day, we have to remind them that it is a community problem affecting even the poor.

Second.—Yesterday, to-day, and probably for several to-morrows, we shall have to remind them that financing a house includes more than first cost.

Third.—We are called on to-day to emphasize as never in the past, that the neglected field of management is of first importance, and that if it is developed as it should be, it will give us valuable knowledge which can be applied in devising sound laws and financial policies.

Because the housing problem is to-day affecting the well-to-do, it is securing an amount of attention never before given it. For the well-to-do are vocal; they speak for themselves without any urging. As a result, there has been a great deal of housing legislation recently, which legislation bears somewhere in title or text the word "emergency", which is not based on any deep study, and which is designed merely to check, temporarily, certain abuses from which a vocal part of the community is suffering.

Such legislation is dangerous. Designed not to cure, but simply to reduce irritation, it is likely to have effects quite unlooked for. Those who live in New York City need not be reminded of some unlooked for effects. Probably no one expected that the Courts would be quite so unable to agree on what is a fair rental, that they would within less than a year be more than 100 000 cases behind on rent trials, and that between $10 000 000 and $12 000 000 of rent money would be tied up awaiting judicial decisions. Whatever one may think of the landlord, however exclusively his sympathies may be with the tenants, he cannot be pleased with these effects, for they interfere with progress toward the only solution that will solve, that is, the erection of an adequate number of good houses.

Of course, some wise people did foresee, if not exactly these particular handicaps, at least some check to building activity because of these anti-rent profiteering laws; they prepared an antidote to be administered at the same time as the poison. This is the law exempting new dwelling-house property from taxation for a period of years.

In New York City, there has developed to an unusual degree the habit of drawing conclusions from local experience. It is found that more homes and more tenement houses have been built in 1921 than in 1920—New York has a tax exemption law applying to 1921 construction—but what about the scores

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of other cities and towns throughout the United States in which were built so much more housing during 1921 than during the year before?

The speaker will read from a letter he has received:

"The name of this city, Columbia, S. C., was not in the published list of cities with good building records for 1920, though that list was taken from information furnished by your department of the Chamber of Commerce of the United States. We, therefore, call your attention to the following facts and ask that you make amends for leaving out of this list the name of a city with a remarkable 1921 record. By August 31st, 1921, the value of building permits issued in this city in 1921 exceeded the total value of building permits issued during the whole twelve months of 1920."

Among the reasons given for building activity in these other cities are the continued pressure of demand, together with considerably reduced prices and a canny suspicion that prices may go up again. Did not these have some effect in New York?

The point in all of this is, that the value of such legislation as described is not proved. The speaker would go further and state that it has been exceedingly harmful in that it has, during these strategic years when public attention was focused on housing, diverted that attention from a real consideration of the subject, a consideration which might have led to results of permanent value, into blind alleys of expediency. It is not wholly flattering to New York to say that it should be the leader in working out a solution of the problem. What is meant, is that New York is so bad—from the housing point of view at least—that it cannot get much worse, and some solution must be found.

It was New York that led the way in securing the kind of regulatory legislation that has permanent value with its tenement house law and its zoning ordinance. These are no mere stop gaps with emergency written into them, but are examples of constructive statesmanship. To be sure they are not fit for adoption by other cities as they stand, for they had to be made to fit the intolerable conditions of New York. They, however, established sound principles of legislation. Other and more fortunate cities can adopt and have adopted those principles to their lasting benefit, by changing the details of standard to fit their conditions. This is the kind of service which it was hoped to secure from New York, and which it is still hoped may be secured before public interest begins to flag.

The tenement house law, fragmentary as it is, for it does not cover one-family and two-family houses or hotels, lodging houses, and similar dwellings, was the precursor of the much more nearly complete housing codes adopted by the more progressive cities and States. Its essential provisions are simple, although the language in which they are clothed sometimes is not. They provide for:

1.—A definite minimum area of open space on the same lot with every dwelling.

2.—Definite minimum sizes of living rooms.

3.—Definite minimum window area.

4.—Adequate and convenient water supply.

5.—Proper toilet facilities.

6.—Possibility for family and individual privacy.
7.—Protection from the weather.
8.—Protection from fire hazard.
9.—Proper maintenance.

The reason for bringing these into this discussion is that many self-styled, hard-headed and practical people believe that such legislation will check building. The speaker believes that the contrary has been practically the universal result. In New York City itself, in New Jersey four years later, in Columbus, Ohio, in Michigan, wherever it has been enacted, such legislation has been followed by increased building activity, for it has made investment more secure. This is a bond that allies legislation with finance.

The enactment of zoning legislation may be expected to have the same effect, although statistics are not available in proof, because most of the zoning laws have been enacted during the abnormal times since 1916.

The enactment of these two kinds of laws and the improvement and greater uniformity in building codes, are aids in overcoming the housing shortage, because they increase the security of investment and, therefore, tend to invite investment money.

As to other kinds of housing legislation of a more direct action variety, there is still too little knowledge to permit of confidence. Three years have been wasted on securing emergency measures of doubtful, or more than doubtful, value, even as emergency measures, and with few or no supporters as permanent measures; perhaps a beginning can be made by adopting certain tests.

In the first place, let people get over the idea that they are facing an emergency in the sense that it is a short-time proposition. With the best of good fortune the obvious housing shortage will not be overcome within the next five years. By this is meant that the vocal well-to-do will not be as adequately provided with good housing in 1927 as they were in 1914. As for the wage-earners, let alone the poor, their problem will be with us a great deal longer. Consequently, it is the part of common sense to base proposals on the proposition that the campaign will be long continued and that whatever is to produce results must be economically sound. These are the tests. During the World War the speaker was an advocate of Government housing for war workers. That was a time of real emergency, and it had a definite terminal point, the end of the war. To-day, the situation is fundamentally different. What is done now has no definite terminal point. Any date that may be set is easily changed, and what is done now establishes precedents and habits of mind which will carry on. Therefore, it is important that these precedents, these habits of mind, have in them the possibility of continued growth and development. Adequate and good housing must pay a fair return on the investment. Only thus can enough good, new housing to meet the growing needs and rising standards be assured.

If this is accepted, there is a wide field for Governmental agencies—which means legislation—that can be no more than hinted at here.

First, real information is needed. A little pamphlet, "The Building Situation", published by the National Chamber of Commerce, with the co-operation of the United States Bureau of Labor Statistics, represents an attempt to secure such information. During 1921, the Federal Department of Commerce has established a Division of Construction and Housing, which is beginning, as has been stated, to assemble and distribute such information. The continuance of its valuable work calls for legislation. This should be supplemented by the States and cities. California and Indiana now have Governmental agencies that will contribute, and other States should follow their lead.

Next, means must be sought for reducing the cost of housing. This will undoubtedly call for legislation. Obviously, there are practices, customs, and inhibitions, due both to law and to convention, that can be changed and, being changed, will reduce costs and at the same time strengthen private initiative or assure fair profits.

Here, perhaps, is the best place to state that, as far as experience goes, such short cuts to the millennium as Government building and management do not promise to reduce real costs. Government operation, not only in the United States, but in other countries, has, as a rule, proved clumsy, inefficient, and expensive. Certain functions must necessarily be performed by Government, but the burden of proof is always on those who would transfer new functions to the Government. Thus far, it seems to the speaker, proof is lacking that Governmental construction or management of housing would produce better or as good results in America as private management. At the same time, however, those who hold to this belief must recognize that dissatisfaction with present housing conditions is causing a growing sentiment for direct participation by the Government. The only effective answer is to produce better results through private operation.

In brief, then, the kind of housing legislation which, because of experience, can be counted on to aid, is regulatory. This kind would protect the man and would contribute to the public wealth from the exploiter who would prey on the public. This kind of legislation: Building codes, housing codes, city planning, and zoning ordinances should be perfected and promoted, because well devised building will be encouraged and investment will be made more secure.

Beyond this, there is a wide field for original work. In this field, however, the notion of an emergency must be discarded and the fact that the problem is chronic must be faced. That being true, proposals must be economically sound, and must have in them the possibility of continued growth and development.

Under the question of finance, certain facts must be faced, namely:

1.—The cost of a house is not its purchase price alone. A house is not immortal, although some owners of antiquated structures would have us so believe. There are as yet no mortality statistics such as the life insurance companies have to guide them along paths that avoid bankruptcy: but it would probably be quite safe to assume that the average life of a city dwelling is thirty years. The cost of the house is its total cost during those thirty years,

with the assumption that at the end of that period it will be scrapped. The speaker admits that this expectation of life is an assumption and that an assumption of 40, 50, or 60 years may be nearer the fact. Whatever the period, however, there should be figured into the cost of the house: (*a*) interest on the investment; (*b*) taxes; (*c*) repairs; (*d*) fire insurance; and (*e*) depreciation and obsolescence fund which will equal the cost of the house. This is on the assumption that the house will be occupied by its owner. If it is an investment proposition, allowances must also be made for: (*f*) periods when the house will be tenantless; (*g*) cost of management; and (*h*) water rent, unless the tenant pays this.

2.—Housing, as a rule, has not been financed on any such basis as that mentioned. Speculation and sentiment have been the mainsprings of action as far as one-family and two-family houses are concerned, and speculation alone as far as multi-family houses are concerned. Most of the housing troubles are due primarily to disregard of these economic factors in housing.

Much has been heard of the difficulty of securing mortgage money. The speaker will merely refer to this difficulty in passing, as an indication of how superficial the discussion during recent years has been, how far short of the root go such proposed remedies as laws compelling certain kinds of banks to loan certain percentages of their funds on dwelling-house property. There was once a time when men of moderate means and men of wealth invested their surpluses in housing property. In the older cities are still remnants of large estates which consisted in great part of dwelling-house properties, groups, rows, or blocks of houses bought as an investment. Along with many other things, these housing estates were pretty well done for during and since the war. The sale prices were too much for human trustees to resist.

The change, however, had begun long before the war. Men were still buying investment houses—the speaker is not discussing New York tenement houses now, but the typical American one-family houses—but they were changing their practice. To illustrate: One such investor would buy a row of new houses from the speculative builder and hold them for ten years. He then would sell to home-owners, and buy another new row. His explanation was that repair bills began to increase after ten years. The home-buyers, being amateurs, did not know this.

This explains a large part of the difficulty in financing. The old-time investor went out of the market, because deterioration in quality of houses was forcing him to become not an investor, but a trader. This deterioration did not affect the mortgage market so greatly, because mortgages usually run only for a short period; they are practically never as long lived as the shortest lived house. Therefore, the mortgagor like the trader, usually got out before serious trouble began. In some places, however, deterioration both in construction and in type was so rapid and so widespread that long before the war certain bankers in New England, for example, began to refuse even mortgage loans on wooden three-deckers.

Here, it seems to the speaker, is a key to the financing problem. Make housing once more a good investment, and capital will be available. The

appeal of housing investments does not and should not be in an expectation of large returns, but in an expectation of sure returns and on security for capital invested. Investors have been led away from this conception by "get rich quick" schemes, by tales of fabulous increases in land values that more than made good depreciation of building values; and so they have come near to wrecking the house property market.

If there is anything sure in this world, it is that if dependence is put on home buyers, not enough houses will be secured. The census figures for 1920 indicate that there actually has been a decrease in home ownership during the decade, 1910-20, despite the housing shortage which forced so many to buy as an alternative to camping in the streets, despite "own-your-home" campaigns conducted on a scale and with a vigor never before attempted.

Honest, substantial construction is essential to success in tempting investment money back into this field, construction which recognizes that the cost of a house is not fully paid until the house is scrapped, that repair bills are an important item, and that even more important is the decrease in value due to the hopeless shabbiness of a "jerry-built" house after it has lost its first freshness.

In the housing codes and the zoning ordinances, municipal authorities are aiming at stabilization of value. These efforts will be useless unless they are supplemented by good construction.

The third and last point—management—will be discussed very briefly. The theory that every one-family house is to be occupied by its owner has played into the hands of the shoddy builder, and has led to utter disregard of housing property management. In the old days, the big estates did develop the principles of management to some extent, but as these estates were split up, as the number of small rental properties increased, the practice grew of farming them out on a commission basis. The results are familiar to all. There is no more generally recognized antagonism than that between house owner and tenant. It has actually become accepted as a law of nature.

Why this relationship should be different from all other business relationships is a mystery to those who believe that it is inherent. It is not inherent; it is simply due to bad management or to lack of management. Along with his belief that his building is immortal, the house owner seems to believe that the house-rental business is the only one that can run itself. He is a belated disciple of the late lamented Keeley who demonstrated perpetual motion. So if success in financing housing is to be attained, we must:

1.—Give housing property a real investment value. This is being helped by housing codes and zoning ordinances. However, honest building must supplement these. Honest building means sound construction plus good planning and adequate provision for such essentials as outdoor space, light, air, and sanitary conveniences.

2.—Begin to study housing property management. Office building managers are working out the principles of their business in their annual conventions, and a few apartment house managers have contributed a little from their experience. As yet, however, the small house managers have not appeared.

It will not be enough to encourage the individual investment buyer to re-enter the field, however. In other lines of business, the individual has been supplemented by the corporation and in the apartment-house field the corporation has appeared. It should enter the small-house field and with its entrance should also enter the field of the tenant stockholder. Other lines of business have found it good policy to have participants become stockholders. Apartment house corporations are trying the experiment. In their case, the speaker is frank to confess that he does not see much hope of substantial and long-continued success. The apartment house has so little of the home appeal, and is so essentially a mere stopping place, that most of the attempts to make it co-operative will prove short-lived. The case is entirely different, however, with the one-family dwelling, for which the prospects are bright, once a traditional attitude of mind is overcome.

PROPERTY IMPROVEMENT AND LANDSCAPING

By Charles Wellford Leavitt,* M. Am. Soc. C. E.

The question of the subdivision of land within city limits after a city layout has been established, becomes one dependent on the desires of those who wish to purchase such land, and on the absorbing power of the purchasers. The inrush for higher pay, to cities constructed for manufactures, drew people from many of the rural districts, and congestion has been brought about thereby. These people will go back gradually to the rural districts where they will work the land and use the habitations. They have started in great numbers already in some parts of the environs of the largest cities; for instance, in the County of Westchester and in the County of Queens, New York State, there is a great deal of building going on at the present time, much of which is on small lots.

The people seem to desire less ground than prior to the World War. Before the war started, some far-seeing people realized that the workingman would leave the city and would demand a piece of land on which he could build a house and obtain the benefits of the country for his children. In one case, described later, a large tract of land was secured in the vicinity of Wilmington, Del., by a philanthropic gentleman. Much of the land in this vicinity is owned in extensive acreages by the powder interests. The whole tract, of about 5 000 acres near the city, was purchased with the intention of giving the workingman a medium sized lot, perhaps 80 ft. by 100 ft., or possibly less, just large enough for a house, and of locating that lot in such a position that the man and his family would have access to park lands laid out in long strips throughout the entire tract.

The land was acquired in a unique way, the purchaser stating to the owners that eventually he was going to buy the land; whether he could do so this year, five years from now, or leave it to be done by others in fifty or one hundred years, it would be taken over, and they could sell it any time they pleased. In order to have it financed properly, a company has been created, holding land that yielded a large income. The reaction of the owners to the proposition was such that they have given over their lands to the company at reasonable prices.

The second great movement, resultant from post-war conditions, is by people of ample means, who wish to live near the city and enjoy all the facilities that the city affords. They are buying lots of minimum dimensions within motoring distance of the city. They do not care to go farther; in other words, as they term it around Philadelphia, they wish to be in "the movie zone". The district lying between 25 and 35 min. of motor travel from the theater district in Philadelphia contains the highest priced property in that part of the country; a district lying 40 min. out, demands almost one-half less. Apparently, 25 min., or so, from the theaters is all that is desired. The same thing pertains to districts near New York City, but to an even more limited

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extent, because the difficulties are much greater of getting back and forth from such a traffic-laden city as New York.

Several illustrations may be cited with regard to the so-called "diagonals" so much discussed. In certain places they seem neither advisable nor necessary.

On Lake Winnipeg, in Canada, lies Grand Marie, a peninsula of the lake, much divided into small peninsulas. The focal point is about in the middle of the largest part. From that, there radiate three distinct tracts of land, with water between the tracts, so that this city which has its head on a promontory projecting into the lake, maintains its street systems, which follow the tracts of land, in three different directions, forming naturally a set of diagonals. The ordinary diagonal of the city, therefore, is eliminated.

A similar condition was found in Kitchener (formerly Berlin), Ont., Canada. The city had been started by several additions which were projected at angles to each other, because the city was traversed by two ravines. There, again, it was practically unnecessary for diagonals.

A few years ago, Long Beach, on Long Island (Fig. 1), was a sand-bar, which, for about 4 000 ft., was taken over by a company and subdivided. The Atlantic Ocean is its southern boundary, and there is a channel in the rear. The railroad has been brought across the channel and is to be extended through the center of the city. There was no particular need for diagonals in this layout. The lot unit, as planned, was 20 ft. by 100 ft., and lots were sold in groups of three. The traffic is largely east and west, so that the east and west streets were placed about 200 ft. apart, and the north and south streets about 700 ft. apart.

In prognosticating traffic, one realizes that if, in the beginning, such a system for New York City had been laid out the present congestion would not exist. Although a diagonal between the Pennsylvania Station and the Grand Central Terminal would be a great help, it would not relieve the situation entirely. It might alleviate it temporarily, but the only solution of the delays in traffic occurring in New York City to-day is the construction of north and south thoroughfares. However, a diagonal is a feature to be respected and not rejected entirely for all cases, although it has been introduced in many places where it was not always necessary, and, at the best, it is a destructive street which becomes extremely dangerous with heavy traffic.

Another large tract of land, which was planned twenty-four years ago and which gradually has been filling up with intensive population, is Garden City, on Long Island, where there existed about 8 000 acres in one tract, a part of which was taken for the subdivision. This tract was traversed by two lines of steam railroad, one on the south side and the other on the north. These two lines have now become practically rapid transit lines, the trains running at frequent intervals into the Pennsylvania Station, New York City, and Flatbush Avenue Station, Brooklyn. It is an astonishing fact that the Flatbush Avenue Station of the Long Island Railroad accommodates more passengers than the Grand Central Terminal, the ratio being about 33 to 28. The stations on the two lines are connected by diagonals, since it was thought that these stations would be the points from which the most intense development would radiate,

these being the two points to which people would wish to go. It has worked out well; these stations have become the nuclei of several developments gradually growing up together as one Garden City. The diagonals are busy streets.

The idea in the Garden City development was for larger lot areas. The project was started by the late Mr. A. T. Stewart as a philanthropic venture, and that feeling permeated the enterprise until 1916, since which time there has been a decided change. The lots were started in units of ½ acre as the smallest, when the first layouts were made, but, during the last five years, these lots have not sold well, and it has been necessary to cut them down to a unit of 120 ft. by 125 ft. or, in some cases, even as small as 80 ft. by 100 ft. In other words, one of the old street blocks, 500 ft. in width, containing lots 100 ft. by 250 ft., was bisected by a new street, which gave practically four tiers of lots instead of two. When that was done, the lots sold readily; people wanted them—they wanted to obtain a lot in the country, with only sufficient room for a house and, possibly, a garage. They did not want gardens, they wanted to live in the country practically as they had lived in the city. It may not be best, but, at the present time, it shows the absorbing power of the people in the vicinity of large cities.

On the west coast of Florida, below Sarasota, there is a condition, similar in a way, to that on Long Island. The land is flat and borders on the open sea. On the Florida seaboard, there usually are keys which form bays between the land and ocean or Gulf, instead of permitting an open coast. This particular stretch of land is acknowledged to be the only point on the peninsula of Florida where the high land (or so-called high land; it is only a few feet above the water) comes directly out to the Gulf waters. The tract is 6 or 7 miles in length by about 2 or 3 miles in width, and has been laid off for future towns, much as Garden City was planned.

The Seaboard Air Line is the only railroad on the property, and it traverses the tract almost a mile from the coast. The main town was projected on the harbor, and at points about two miles apart were located future centers which were connected with the railway stations and with the coast, and cut across by diagonals so that the people could get back and forth from their points of business to their homes, and also to the water and to the line of transit.

The problem there was to care for the people who go to Florida to make a living. They wanted larger places so that they could develop the industries of raising vegetables and fruits. In addition to the water-front developments, there were about 250 000 acres to be developed in the back country, where the land is divided into farms, with five acres as the unit and where, since people have been arriving in great numbers, the land has advanced in price from a few dollars to several hundred dollars per acre. People who have gone to Florida to live, have discovered that about five acres of land may be devoted to the raising of fruits, without much extra help; that is, one man could do the work and make a living. Therefore, behind the highly developed tract on the water, which is mainly a winter resort, there are large tracts, divided by diagonal roadways, devoted to farming. These roadways consist of two strips of concrete, 18 in. wide, which are used as motor tracks. They are practical

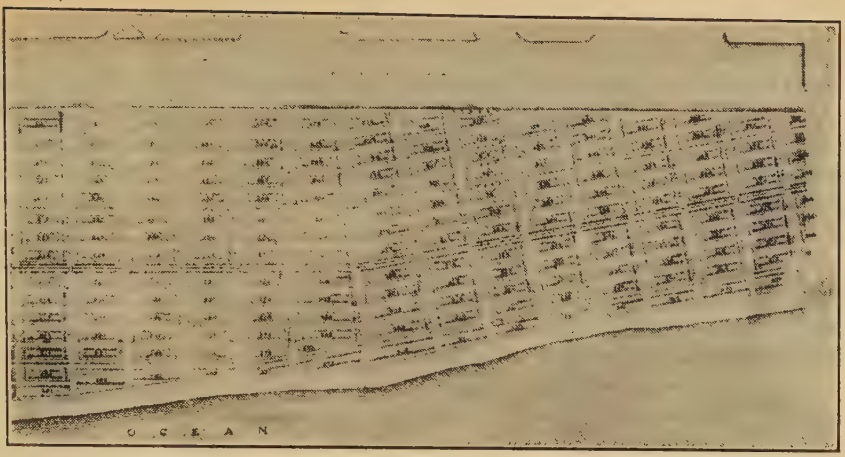



FIG. 1.—LAYOUTS AT LONG BEACH, LONG ISLAND, N. Y.

FIG. 2.—PENNYPACK PARK PROJECT, PHILADELPHIA, PA.

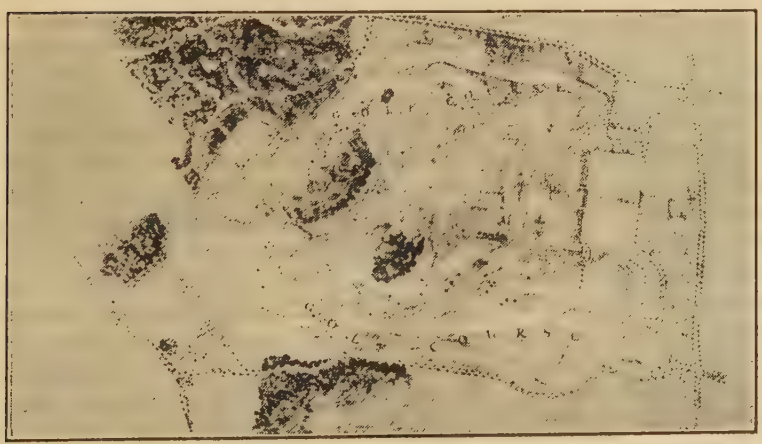
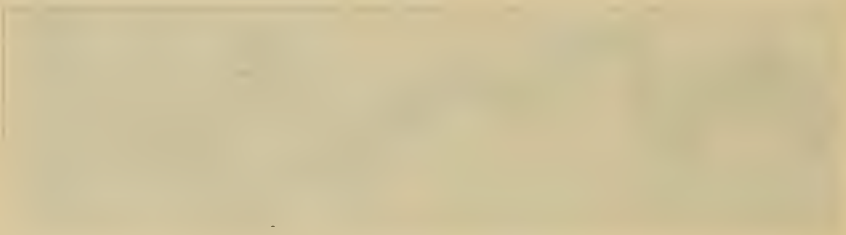


FIG. 3.—WESTCHESTER BILTMORE COUNTRY CLUB, RYE, N. Y.



and are much used in that part of the country. When the automobiles meet, they turn out on the sand which will bear the weight of a car once or twice; after that, it becomes pulverized and driving is difficult.

Pennypack Park, in the City of Philadelphia (Fig. 2), presented a different question; its location in relation to the city, and its construction, were difficult problems. The layout of the streets around the Park was difficult to work out. It seemed wise to border the Park with a street, instead of, as in many cities, backing the lots on the Park. A marginal street, therefore, was built around the Park, as in the case with Riverside Drive and Central Park, in New York City. Pennypack Park has an unusually circuitous route, systematically treated with marginal streets, to conform to the street layout of this section of the city, under the definite city plan adopted.

It is extravagant to have lots face on one side of the street only, but, on the other hand, the Park and also the lots are benefited. The reaction to the entire tract probably is much better than it would be if, as in many real estate subdivisions, the streets were made so that the lots would be on both sides, with the backs of the lots against the Park. In this case the subdivision of the lots will be city lots, probably the regular Philadelphia unit of 25 ft. by 100 ft. It will be an intensive development, and the Park affords the only means, in that section, for relief from the congested condition of the city. The Delaware River is on the extreme east, and the City of Philadelphia owns a table-land between the river and the beginning of the Park. Pennypack Creek winds "S" shape through this land and, sometime, these lands will be developed as a city.

The unit of subdivision should be small where the city builds a park. Where no park is built, there is need of a large unit for the lot. The question of air and light in the suburbs of cities has become serious. The County of Westchester, New York, has appointed a Commission to make a County Plan. When a problem of this kind is taken up, and the layout is discussed, it is found that the establishment of parks is a most momentous question. People do not care to give up land, and the only solution is to develop parks on the lower priced land; where this is impossible, the numerous golf courses surrounding the cities are likely to solve the problem.

Golf courses largely are owned by golf clubs, and may be the great salvation of the future in the development of the environs of a city. Even if they become valuable, it is quite probable that the municipalities will take them and develop them as parks when the clubs can no longer carry them.

One of the most recent developments in Westchester County is the project of Mr. J. McE. Bowman, the Westchester Biltmore Country Club, at Rye (Fig. 3). This project required about 600 acres of land and three golf courses were established practically in the center of the property. The acreage is on a great elevated plain, about 150 ft. above Long Island Sound, which lies to the south and east of the property. The Club House which is near the east border, will have about 400 rooms and is being built at a cost of about $4 000 000 or $5 000 000, in addition to the cost of land, which, was nearly $2 000 000.

Three golf courses which are the main object of this club, radiate from it. There are also tennis courts, riding schools, and polo grounds, and the other

facilities that go with a country club. The land between the edge of the course and the margin of the property is subdivided into plots ranging from ½ acre to 2 acres, which are selling rapidly. Although the property is 25 miles out from New York City, these plots are being readily absorbed, the idea being that the golf courses will form a residential park for the purchasers. The land is partly wooded, and partly in the open, but the size of the lot is a minimum for the class of people who are buying.

The people who are buying lots of ½ acre to 5 acres, formerly bought tracts of land from 200 to 500 acres in extent. By having the small lot, with all the improvements that can be given, and the facilities afforded by that large tract of golf land, riding grounds, and other amusements, they have solved, to a certain extent, their problem of life. The speaker thinks the future of the project is likely to be good; a large tract of land held in that way is most desirable.

During the World War many of the industries of the country had to have people near them. The Midvale Steel Company built a village to house its employees. The only available land near-by was a steep side-hill, which was a difficult place to develop. All the industries along the Brandywine, in Pennsylvania, are in the bottom of the valleys and, in this instance, a diagonal road, on a fairly steep grade from the lower level to the upper level, was the only way to get up the hill. The streets were laid out from that road, following the contours, the idea being simply to give the workingmen the smallest piece of land that would afford a house which would not be a block house, as they wanted their individual homes. The plans were developed in that way, and the houses were built, and this development became a permanent part of the Town of Coatesville, Pa. It was not just a village that was built to be destroyed after the war, as many were, but it was a permanent addition to the town. The houses were built about as close together as possible. There was no attempt to have gardens, the people did not seem to want them; they wanted just a place where they could have a house—some of them who became fairly well-to-do during the war, had a garage.

The layout near Wilmington, previously mentioned, is practically an extension to that city. The Brandywine is a circuitous stream, which makes its environs the more interesting. The northern boundary of this development is about fifteen miles below Coatesville, and the City of Wilmington is on the south. The tracts of land that were taken for the addition, are on the east side of the Brandywine. Between the city and this tract of land is a large property which could not be obtained. Part of it is owned by the city and is used for reservoirs, and a part is owned by an interest which probably will sell at a future date. To the east is the intensive development of these 5 000 acres lying near the Brandywine. The land is called Brandywine Hundred, as in the original deeds given by the King. Through the development are parks and diagonals connecting the various centers with the outlying district. The City of Wilmington is an old-fashioned layout, being one of the first developments in the United States. It has gradually been expanded with the thought of fairly large city lots. That motif will be carried through this new

development; instead of city lots, they are about 75% larger in Wilmington than they are in New York City or Philadelphia.

The desire for a small lot, about 25 to 35 min. drive from the city, is illustrated by the development planned for a piece of land at Bryn Mawr, which is in "the movie zone", just outside of Philadelphia. After careful study, it was found that the entire surrounding country was intensively developed, largely with good-sized houses occupying almost the entire lot. In the suburbs of Philadelphia, the houses are from 50 to 75% larger than houses in the suburbs of New York, and are much better built. It was found, however, that nearly all the owners of houses had sold all the land except enough for the house and a garage, but no garden. The development at Bryn Mawr, therefore, has been laid out to give to each lot only sufficient land for a detached house and garage, and practically nothing else. Formerly, in developments, it has been felt that gardens were necessary, and it still may be so, but people do not want them. If land is to be cut up for disposition, it is impossible to sell more than is wanted at the time of sale. In a few years, when the property is embraced within the city, the area of land will be reduced, the houses will be torn down, and block houses will be built. Two of the present lots may afford room for three houses, so that it is not only wise, but positive economy to place these streets at about the right distance apart for the future development of the city, which will embrace, eventually, these suburban territories.

HOUSE DESIGN

By Andrew J. Thomas,* Esq.

Whether one deals with the City of New York, or any other city, housing in tenements is the same. During the speaker's study of this work, many complex problems have arisen, which pertain not only to technical matters of construction and of plan, but also of convenience, finance, economics, and, among others, the most important of all, the human element.

Owing to the high cost of building, the speaker has had occasion to study the economics of the situation. Fig. 4 shows two plans which demonstrate what is meant by economics, bearing particularly on zoning, light, air, and ventilation. These two plans occupy corner lots, 100 ft. by 100 ft. in size; they each have rooms of approximately the same size, with 32 rooms to a floor, and, therefore, they furnish an unusually good unit basis of comparison.

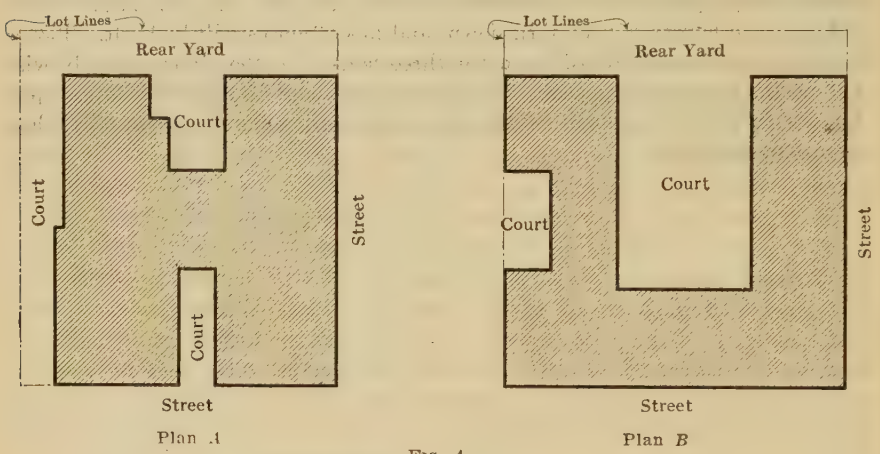


Fig. 4.

Plan *A* was adopted by the speaker before the World War, and it is possibly the first of its kind designed and carried out in a constructed building in New York City. It was extensively copied and, therefore, it may be said to be a type plan in New York City. It covers 7 900 sq. ft. of building area on the 10 000 sq. ft. of the lot, or 79% of the site. Plan *B*, however, covers only 6 200 sq. ft. (62%), or 1 700 sq. ft. less than Plan *A*. Since both plans have the same number of rooms and the same rentable area, it is obvious that the 1 700 sq. ft. of excess floor area in Plan *A* over Plan *B*, may be looked on as wasted space.

There are other points in favor of Plan *B*. Its rear rooms are lighted by a court, 37 ft. 6 in. wide, which width is regarded by the Tenement House Department of New York City as being almost a garden. In fact, it ceases to be a court. Plan *A* has courts only 12 ft. and 20 ft., approximately. How

* Archt., New York City.

much finer is the light, air, ventilation, and cross-ventilation and—the human factor, the livability—in Plan *B* than in Plan *A*! Furthermore, with Plan *B* the Tenement House Department has granted permission to remove the fire escapes from the street front of the building and to place them in the court, whereas the law states that any fire escape with exit leading to the street, shall be placed on the street. This street location of fire escape is a characteristic of Plan *A*. Because of the great width and the superior openness of Plan *B*, the authorities have granted the concession of placing the fire escape in the rear. To the speaker this seems a great advance in housing planning, because it takes from the city streets the unsightly fire escapes which are an encroachment on the sidewalk and a detriment to real estate values.

A peculiar situation with regard to Plan *B* is that, after this saving in planning has been made, by squeezing out of Plan *A* about 1 700 or 1 800 sq. ft., which means a saving of, approximately, $40 000 or $50 000, based on $0.40 per cu. ft. of cost, the financial interests in the City of New York will not lend as much money on Plan *B* as on Plan *A*, notwithstanding the greater economy of plan and construction of the former, and its far better conditions in respect to light, air, and ventilation, and removal of fire escapes from the street. Plan *B* has the same, or perhaps a little greater, gross rental income, due to its better situated rooms, a $40 000 less construction cost, and a much lower maintenance and depreciation cost. It is a much better business proposition than Plan *A*, yet it is penalized on a loan. This seems a strange condition, and the speaker feels that it is his duty to point out to the members of the Society this drawback in housing.

After Plan *B* had been developed, the first man to come into the speaker's office was told of the new plan and what it could do for him. He said, "That is all right, if you can show me a saving of $40 000 in a structure of that sort, I shall be very glad to pay you $1 500 or $2 000 extra for your plan, but will the loaning interests give me the same amount on that plan as on the old one? Without consideration, the speaker replied, "Certainly, they will".

When the new client had gone, the speaker began to realize that a building loan is appraised on a basis of the cubical contents of the building. The size of the mortgage is determined by taking a fixed percentage of the valuation of the building, as established by multiplying its cubical contents by a unit figure of building cost per cubic foot. Take, for example the cubical contents of the two buildings represented by Plans *A* and *B*, for a building five stories high. It will be found that the loan on Plan *B* will be more than $20 000 less than that on Plan *A*, regardless of its superiority in economy and desirability. The financial institutions place a premium on lost motion and waste space in a building.

When the speaker discovered this unfortunate condition, he interviewed the heads of the loaning institutions and was told that they could not give the same consideration to the improved plan. When he expressed surprise at this, they told him that the only way they knew of appraising buildings was on the basis of full cubical contents. The speaker said, "Gentlemen, do you realize that you are creating a future slum?"

Without exception, Plan *A* represents the future slum of New York. That little court, 12 ft. wide, is not adequate. It causes darkness, odors, dirt, rubbish, and other unsanitary and inflammatory conditions. Its greatest weakness, however, is in the matter of maintenance. Such a court is difficult to keep clean. Tenants, landlords and employees will allow dirt and rubbish to accumulate in the court or on the fire escapes, and, in the dim light, inspection is difficult. The experience of the City and Suburban Homes Company of New York, which, for many years, has successfully housed about 10 000 wage earners at a rental lower than the speculative tenement builder can offer, is that narrow courts are so difficult to maintain that they are a weakness in a tenement plan. Whereas, with wide courts as in Plan *B*, housekeeping is exposed to the light, and to the public gaze, and fear of public opinion, if nothing else, induces cleanliness.

Plan *A* is the type fostered by many financial interests, owing to lack of knowledge of conditions of tenement housing. However, the speaker has asked the New York Chapter of the American Institute of Architects for its support in upholding sound principles of appraisal. After several conferences, the President of the Chapter appointed a committee to discuss the matter of appraisal with the loaning institutions of New York City. This committee is developing a model tenement for New York City.

Another point remains to be considered in this complicated matter of tenement planning. This is the percentage of lot occupied by the building. In New York City, the law allows—together with zoning restrictions as to courts, which the speaker feels are too lenient—70% of the land to be occupied on interior plots and 90% on corner plots. The tendency on the part of investors has been to steer as close to the law as possible. The present laws and the building world, however, recognize the necessity for economical construction and for admitting light, air, and ventilation into the building. Opinion differs now only as to how far land values permit such improvement. Outside of Manhattan Island, and on land costing $1 500 to $5 000 or more for a 25 by 100-ft. lot, it is generally conceded that buildings are not economical when they occupy more than 50% of the area. The speaker designed a group of four-story apartment buildings in Brooklyn, N. Y., which occupy the end of a city block, 200 ft. along the avenue and 171 ft. along each street, and house 96 families in three and four-room apartments. Each apartment has two or three exposures, almost like an individual country house, and overlooks, not a court, but an outdoor garden. This group of buildings occupies only 44% of the lot area, and the owners calculate that it can be rented about 19% cheaper than more congested types, like Plan *A*. The superiority of this Brooklyn group is due both to the more economical disposition of floor area, not possible in a more congested and more complex plan, and to much cheaper maintenance. The four buildings are operated by a force consisting of a superintendent, two men who tend the central heating plant in cold weather and work outdoors in warm weather, and three women who do cleaning. The buildings are kept clean without great effort, apparently. By merely taking a few steps from a point in the middle of the group, one can inspect every court and area and fire escape of the four

buildings. In this case, bad housekeeping would be in the view of the public, and, consequently, does not appear. Easy maintenance, in which the co-operation of the tenants is induced by proper design rather than by irksome and expensive personal supervision by the management, causes less depreciation.

For the reasons of maintenance, together with high cost, due to excess space and to lost motion, the stereotyped apartment tends to become slums. It is too costly and, in addition, the owner, vainly struggling with the heavy depreciation, cannot rent his property at a figure high enough to allow for proper maintenance, and it is permitted to degenerate into slums. During the period of depreciation into a slum, the tenement is apt to pass through one or more foreclosures. Such is the history of many tenement properties.

Tenement house management is a business in itself, and the reason for its backwardness is that heavy depreciation is laid to carelessness of tenants, whereas careless planning of the building may be blamed. One can deal with slovenly tenants, but nothing can be done with an inefficient building. Defective plan arrangement promotes slovenliness in tenants.

As a result of a growing appreciation of the importance of these factors of maintenance and their relation to depreciation, the difference of opinion still existing resolves itself into the question of how far congested types of plan are necessary on the highest price land, such as the crowded areas of Manhattan Island. The working classes, obviously, cannot be housed in high "elevator" structures, and the five or six-story "walk-up" is their only salvation. The speaker has demonstrated, to his own satisfaction, that the maximum area that can be covered economically in interior lots is between 50% and 60%, and only a little more than 60% on corners, very small frontages being disregarded in this comparison. Plan *B* covers 62%, and has the same number of rooms of the same size as Plan *A*, which covers 79% of the land, and it can be built for less money. One can still reduce the size of living rooms and bedrooms, and thereby increase the number of rooms, but not many more—and still keep them fit for human habitation. Satisfactory housing is impossible as long as bedrooms of the minimum legal size allowed in New York City are permitted. A bedroom 7 ft. wide, to accommodate two persons during hot weather, does not promote efficiency or happiness. The speaker feels that studies now being made prove that for those who can afford to live in new housing on expensive land, the most economical housing is in tenements covering from 50 to 60% of the area of a 100 ft. by 100-ft. interior lot, which contain as many as 42 rooms per floor, in three, four, and five-room apartments. The room sizes are naturally small, but may still be within the bounds of decency. Such a high number of rooms per floor, however, presupposes elimination of excess space and lost motion, such as is hardly ever attained in this class of planning. Even with 42 rooms per floor, 6 of them will face on a side court 6 ft. wide, along the party wall extending from street to rear yard, a condition which will give fairly good ventilation, but not much outlook.

In conclusion, the speaker believes that if all would join in the movement to further sound planning and finance, it would strengthen the zoning principle. If by zoning laws a better and more efficient planning of streets and cities is effected, of what good is this work if a type of house such as Plan *A*—uneconomic and a future slum—is going to be built? In the larger cities, with high land values, the speaker believes the only economic way to house the people is in multi-family types of dwellings.

CERTAIN ASPECTS OF THE NATIONAL HOUSING PROBLEM: PLANNING AND ZONING

By Morris Knowles,* M. Am. Soc. C. E.

The National housing problem still exists, even though industry is rather quiet, and the cities are not attracting as many people as during the World War. It appears, however, that many people, having entered the field of industry on account of high war-time wages, now dislike to return to the rural districts after having obtained a taste of city life. The problem of housing these additional workers was felt so keenly during the war that the Government designed and built many new and complete towns. This helped to relieve the situation to some extent, but was insufficient to make much of an impression on the National shortage, caused by a general lessened building program.

When factories are again operating on a 100% basis, the housing shortage will be felt more keenly, especially in industrial communities, where it will be necessary to provide homes for employees. The necessity is obvious, since, during the war, a lesson was learned from the keen competition resulting from labor flocking to those industries having more attractive housing developments and better facilities for living.

Fundamental Considerations

One answer to the question of how to solve the housing shortage, is to provide more houses. More houses require more building lots and these, in turn, require more sub-divisions, more streets, more utilities, and miscellaneous facilities.

Housing developments may be laid out as: (1) New and complete villages; or as (2) additions to existing cities and towns.

The first will obtain in connection with new industries to be located at some distance from already established communities. There is no definite assurance, however, that industries will be in a position financially for some time to come to embark on extensive housing undertakings, and, on the other hand, there is some tendency for factories to establish themselves on the outskirts of existing cities. It would seem, therefore, that the most pressing problem of the immediate future is that of providing proper and adequate additions to already existing communities.

With this tendency to increase the population of established cities and towns, it is well to consider the effect of housing operations on the community spirit and life. In a general way, the greater the number of home owners, as distinguished from transients and rent payers, the more can one expect sound industrial relations, civic pride, and community progress. If these advantages are to be gained, the home purchaser must be offered the attractive surroundings resulting from proper planning and the security against undesirable encroachments provided by zoning.

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Successful housing—in the larger sense of the word—requires careful consideration of such phases of city planning as:

(1) Topography, natural resources, and facilities.
(2) Traffic, transportation, and street facilities.
(3) Public utilities and public works.
(4) Areas for residences, business, and industries, that is, districting.
(5) Areas for parks, playgrounds, schools, cemeteries, and public buildings, etc.
(6) The protection of these areas by zoning regulations. In other words, the actual planning and zoning of housing developments is a matter pertaining directly to the general field of city planning.

Whether such developments are made as separate communities, or as additions to existing cities and towns, they require careful consideration. A definite and orderly study must be given to each problem, incident to the evolution of a comprehensive plan.

There is an opportunity for accomplishment in the planning of an entirely new community, which is not presented by ordinary sub-divisions or real estate developments in connection with older municipalities. Many of the factors, such as the character of the buildings, or the amount and movement of traffic, which are frequently problematical in the latter instances, can be made somewhat more determinate in the planning of an independent town. The new community can be planned intelligently, therefore, to meet definite requirements and conditions. Again, the problems and the order of their consideration will be quite different than in a revision or re-planning of older communities.

Where home sites are to take the form of municipal sub-divisions or real estate allotments, however, they must be designed so as to fit into the comprehensive plan of the city. In other words, consideration of the larger community as a whole must be made, in order to plan intelligently the new housing developments. Thus, the proposed traffic arteries will be contiguous with existing ones; schools, parks, playgrounds, civic centers, etc., will be provided where needed, and, in fact, all the facilities will be an integral part of the whole working plant—the city.

Consideration of the Plan

There are two necessary closely related steps which have been found advisable to consider together, after the general requirements of the housing project have been determined. These are: (1) the main lines for traffic and transportation; and (2) the sub-division of the land into the various areas best suited to their prospective uses and the regulation thereof by zoning laws.

It has been too often the case that real estate allotments have been laid out irrespective of transportation requirements, public utility necessities, and arrangements, and the areas best suited to business, industry, residence, or parks have been ill chosen. These should be chosen with respect to the possibilities of economical development and their "fitness to function".

One of the important needs is sufficient legislation by which a plan or layout for housing sub-divisions may be made effective and not shifting or

changeable. In addition, laws providing for district, as well as abutting, assessments for improvements and laws for excess condemnation and fixing of future street lines are all helpful and necessary for completing the fulfillment of adequate residential areas.

Traffic and Transportation

The sub-division of the land should be accomplished on a basis of transportation requirements, that is, the character, volume, and destination of traffic, and as influenced by topography. Traffic and topography have an important relationship to the location and direction of streets, their width, and grade. Traffic thoroughfares should be located so that they will connect the various centers as directly as possible. Modifications and deviations from the straight line will necessarily be made, in order to keep cuts and fills within reasonable limits and to prevent damage to adjoining properties. In locating traffic thoroughfares, the relationship to the sub-division of property is of relatively minor importance.

Secondary thoroughfares, however, should be planned with some reference to the sub-division of property, particularly to avoid division into awkwardly shaped lots. Directness, nevertheless, should not be unduly sacrificed, although it is not as important as in the case of arterial thoroughfares.

The planning of residential streets is more closely connected with property sub-division. The principal purpose of such streets is to provide access, vehicular and pedestrian, to and from the houses; and to afford an open space between the houses, thus providing light and air. The relation of the residential streets to the primary and secondary streets of the housing village is of great importance. It is desirable, for a number of reasons, to plan the former, and their connections with the secondary and major streets, so that a free outlet to traffic will be afforded, without attracting through traffic and heavy hauling into the residential streets. This is not to be accomplished so much by imposing objectionable difficulties in the way, such as excessive grades, too narrow widths, or by kind of paving, but by inviting traffic to the main highway, by the superior facilities and convenience afforded. Such regulations of traffic can be further affected by making traffic routes through the residential streets relatively indirect, compared with those maintaining on the main highway.

Zoning and Districting

Effective planning means little if its beneficent results are not protected by zoning regulations. The districting of the city as a whole, and the limiting of various classes of business and industry to certain areas, give a permanence of plan and a protection against encroachment, which should be assured to the home owner.

With the primary districting plan well established, it becomes desirable to provide various residential sub-districts, dependent on the contemplated size of the project and variety and type of houses called for by the character or classes of people to be housed, the required size and shape of lot, and the cost of the land. Districting consists, primarily, in utilizing the various parts of the town and parcels of land in such manner as to serve the health,

safety, and welfare of the community to the best purpose. Furthermore, it should include the definition of restrictions or zoning, to establish the districting policy and to insure permanency in the use of property. It is necessary, not only to make the most effective use of the property and to build on the most adaptable ground, but also to protect the future purchaser.

If zoning regulations are promulgated and enforced from the beginning, they will define the development of districts for many years to come. Such regulations are now being enforced in a number of American cities and are being upheld by the Courts. They insure that the purposes of the development will be obtained and, at the same time, protect the interest of the community and the individual. It is of interest, therefore, to note that the United States Department of Commerce, in the wisdom of its Secretary, Herbert Hoover, M. Am. Soc. C. E., has planned to promote intelligent and wise consideration of the subject, by the dissemination of information directed to call attention to the important and necessary details of this subject.

Such regulations, although an exercise of the police power, must necessarily be based, in the first place, on careful designing and the study of probable use. The separation between various districts should not be made too evident, in order to avoid the creation of a prejudice against the property of lower value. A watercourse, ridge line, woodland strip, parks, or other topographical features may be utilized as a desirable means for the purpose. The various residential districts should preferably be contiguous, thereby reducing the outlay for schools, fire protection, and, generally, the cost of utilities.

Among the various areas desirable for different purposes in a new town, or in addition to an already existing city, may be mentioned the following: (1) Industry; (2) business; (3) residences; (4) civic and community centers; (5) schools and playgrounds; (6) parks and parkways; (7) churches and cemeteries; (8) public and semi-public areas, such as, hospitals, children's homes, etc.

The extent to which the foregoing areas are to be introduced into the plan of a housing development will depend largely on its size and location as well as its relation to other and adjoining communities. The various elements and their underlying requirements heretofore noted are more or less interrelated; and their incorporation in a town plan is the problem of co-ordination and adaptation. Too much emphasis cannot be laid on the importance of consistent and co-ordinated planning; on the necessity for careful consideration of the essentials of each element or feature; and on the merging of the whole into a well-balanced program.

Residence Areas

The number of dwellings per acre, or the building density, shows the degree to which the property can be occupied and affords the common basis of comparison of use. It is, therefore, the measurement of the saturation of the plan and also an index of housing conditions. This is best expressed by the number of families housed per gross acre, including the street area, but excluding parks and open spaces. In any particular location, the greater the number of families housed per acre, the less the cost per unit, which

will result from the plan. A high density, however, brought about by crowding a large number of families on small lots, with narrow streets and lack of open spaces, is poor economy and bad design. It leads to undesirable living conditions, the correction or prevention of which is the object of planning and zoning.

Density will be influenced by the type and grouping of houses, the width of streets, and the space allowed for front yard, back yard, and between houses, rows, or groups of buildings. A comparison of building densities will indicate the real situation only in a general way, as the disposition of the open space provided and the degree in which it is useful, are as important as the amount. Detached houses, placed closely together, may afford an amount of total open space nearly as large as row houses, but the side-yard space may not be useful in adding to convenience and in providing necessary light and air, in fact it may be detrimental. When the cost of land is high, the number of families housed per acre may be increased, and this can be done best and most attractively by building row houses, apartments, or two or four-family houses, rather than by crowding detached or semi-detached houses on small lots.

The simple but often neglected relationship between streets and grading and house location in a residential sub-division will serve as an illustration of the interdependence between the different items of the plan. Streets should not only be located and graded so as to fulfill the requirements of traffic, access, and drainage—their prime functions—but they should be also fixed with proper regard to economic and desirable house locations, in order to minimize the cost of lot gradings. The excess or deficiency and the proposed disposal of materials from street excavation may well be a factor in the development of design for lot grading. Also, the use and location of alleys, with their attendant expense of construction and maintenance, will depend very largely on the type and grouping of the buildings. Illustrations, such as the foregoing, could be multiplied indefinitely, showing the necessity and practical benefits obtained by the working out of a carefully considered and comprehensive plan before beginning work.

It is necessary, therefore, that the streets and lots should be studied and located as a unit. The designer must constantly have in mind how the lots may be developed, the approximate elevations of the houses, the relation of the front yard and the street, and the possibilities of private driveways within the lots for use of automobiles and service wagons. It has been too often the case that streets have been located with regard to cuts and fills within the street lines only, leaving the property owners on either side to develop their lots as best they may.

Street gradients should be established on a basis of their effect on the abutting lots as well as the necessity of maintaining minimum street grades. Improper street gradients often produce such undesirable lots as to make them almost unsaleable. The relationship between existing and proposed gradients of streets located parallel with the slope may be quite different from those streets laid out at right angles to the slope.

The utilities necessary to serve the community must be considered as a fundamental problem in connection with the layout and, therefore, must be

studied at the same time. In addition to being carriers of traffic the streets may also serve as the convenient locations for utility lines. Drainage and sewerage can be effected far more economically if reasonable consideration is given to the requirements of lines and grades of streets. The location of a street in a valley, in such a manner that it unnecessarily crosses the drainage line, thus introducing rising grades, may require excessively deep cutting for the sewer. Again, it may be possible to locate a cross street on a line that will not only serve the requirements of traffic and access, but also afford the best available location for the sewers and drains. If this cannot be done, it may be necessary to undergo extra costs to lay the sewers and drains on private easements, or to follow an indirect route, both of which are objectionable for many reasons.

Conclusions

It is desirable to emphasize the necessity for a comprehensive study and plan for all village developments. The construction of housing projects properly planned does not cost any more than for those constructed with little or no planning. In fact, foresight in planning for future requirements often saves the community many thousands of dollars in the cost of street widenings, street maintenance, condemnation of land and dwellings for parks and playground purposes, and for many other items.

There is a definite need of education and appreciation by all as to the economics and advantages to be gained by having housing projects properly laid out and developed and thus protected by adequate zoning regulations in order to promote the health, safety, and welfare of the community.

The housing problem comprehends not only the need of buildings, but also an opportunity for community progress. Village projects should be conceived from the broad view of the city planner and gifted realtor; executed with the experience and skill of the engineer, architect, and landscape architect; and made permanently desirable by zoning.

TRANSPORTATION AS IT RELATES TO THE NATIONAL HOUSING PROBLEM

By Ernest P. Goodrich,* M. Am. Soc. C. E.

Until within a few years, the improvements secured in the housing situation were almost exclusively through the creation and enforcement of tenement house laws. The possibilities of improvement were greatly increased when the power of zoning was given communities, beginning about 1916 in New York City. Since that date, enabling acts have been passed in many States and the zoning idea has spread rapidly. The Hon. Edward M. Bassett, one time member of the Public Service Commission of the First District of the State of New York, and, later, Chairman of the Commission on Building Districts and Restrictions of New York City (the Zoning Commission), is authority for the statement that the enactment of the zoning ordinance of New York City, which cost for its preparation and promulgation, about $75 000, has been of as much benefit to the community as the $350 000 000 rapid transit system. This statement may be interpreted as depreciating the subject of transportation, but it is believed that scientific zoning will go far toward alleviating present-day transportation problems, especially as they apply to the needs of the worker for transportation between home and office or shop.

Through zoning it is possible to plan cities so that the worker may be able to live near his work in far greater numbers than has usually been possible in the past, by fixing the boundaries of commercial and residential districts and their relative areas. In smaller cities, and in sub-divisions of larger cities, it is quite within the realm of possibility for practically all the people to walk from home to office or factory and to visit the local store, theater, or school without having to make use of rapid transit facilities. Resort has been made to several plans to secure this condition. Perhaps the best is exemplified in the zoning plan devised by Mr. George B. Ford for Rheims, France. The authorities of that city first adopted a typical American gridiron street plan for their new city. It was pointed out to them that they were losing their personality as a community and that the best American city planning practice included zoning in such a way as to bring about the best relationship between the industrial and residential districts. The city authorities abandoned their first plan, and eventually adopted the one devised by Mr. Ford. Industrial districts which were wedge-shaped, were located radially about the city, with belts between adjacent wedges in which residences should be located, together with stores along the main radials which traversed the residential belts toward the principal mercantile district at the center of the community. The residential belts were approximately ½ mile wide, so that a worker need walk no more than ¼ mile from home to factory and a much less distance to do his shopping.

If it is assumed that a residential lot is 40 ft. by 100 ft., that 40% of the residential district is devoted to streets and other public purposes, that one worker with his family occupies each lot, that for each worker 400 sq. ft. of industrial area is required, and that no worker is to walk more than ¼ mile

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from the boundary of the factory district to his home, then it is an easy matter to compute the diameter of a theoretical industrial district which would house a corresponding number of families of workers in a belt ¼ mile wide around it. It is also interesting to note that one side of the circular thoroughfare which may be assumed to surround the industrial district, would provide ample frontage for all the usual and necessary mercantile establishments required to provide for the 3 000 workers for which the district has been assumed and for their families. By arranging a new city or re-arranging an old one, it thus seems possible largely to cut down transportation facilities.

Much progress has been made in improving transit facilities so as to carry workers considerable distances to and from work. The provision of improved facilities tends to create suburban centers with intervening, unimproved districts between them and the central part of the community. These unimproved stretches are burdens on the municipalities. Utilities like gas, water, light, and pavements must be extended into the suburbs, the mains and feeders being designed of sizes sufficient for large future increases in use, on all of which, interest and maintainance charges must be paid. The better the transportation facilities, the larger, generally are these vacant stretches and the heavier is the community burden. A curve might be drawn which would indicate this condition, in which the ordinate would increase with distance from the center.

Congestion of population is equally a municipal handicap. The wider it can be spread, the less is the community cost in the way of sickness, immorality, and the other evils of congestion. A curve portraying these elements would have a downward slope as the distance increased from the center of congestion. Were these two curves superposed, the ideal condition would obviously be the point where they crossed. In other words, just sufficient transportation should be provided to obviate congestion, but also only sufficient to create as compact a community as is economically possible.

The decrease in congestion as one moves away from the center of a community is reflected in real estate values. The latter seem to be inversely proportional to the distance traveled from the community center. In an investigation made with reference to a certain district in Brooklyn, N. Y., it was found that the time zone map and the real estate valuation map on which equal-value lines had been drawn, were strikingly similar. It was ascertained that 1 min. travel on the rapid transit system corresponded with a $20 per front foot of interior lot decrease in real estate value. Assuming a 25-ft. lot at $20 per front foot, with one family per lot, from which the man rode twice each week day and his wife took four rides per week, it was found that the value of the time of these individuals was 4 cents per min. This corresponds with an annual income of about $5 000 per family per annum, which was known to agree well with the class of people who reside in the district of Brooklyn, which was studied.

All are familiar with what was done during the World War in extending residential zones around war plants, by developing transportation facilities. Except for war needs, such a process is not the best for any community; however, with large plants, this condition becomes imperative. While sitting beside a worker in a street car in Detroit, Mich., it was ascertained that this

man consumed 3 hours each day traveling between his home and the Ford Motor Plant in which he worked. He thought he was exceptional, but believed that the improved conditions, with open air and country, provided for his family, were better for them than to live amid greater congestion so that he could spend less time on the street car.

A questionnaire distributed in Cincinnati, Ohio, revealed the fact that among 1 000 workers, there was a preference on the part of 90 to 95% for homes in the suburbs. The answer to the question as to why they held this preference gave an almost equally large number of answers to the effect that desirable housing facilities were not available in the down-town district where they would otherwise prefer to live. There is thus seen to be a large element of psychology even in the transportation business as it affects housing.

Concentration is being superseded by dispersion. The old-time central meeting point, to-day is being disseminated. Every school is being equipped as a social center and many community sub-centers are being built, so that there are only the larger theaters, the larger department stores, and the railroad station that are in any sense unique centers, and they can be widely separated to great advantage. The banks are opening branch banks; the post-offices have sub-stations, and many stores of the chain type have widely distributed branches. There are the corner grocery and the corner drug store. In some cities, it is even possible to pay one's taxes without going to the City Hall. Such dispersion is a great reducer of transit congestion.

As far as the congested districts in cities are concerned, a tendency seems to be growing toward the elimination of street-car tracks in down-town areas. The present tendency is toward the elimination, as far as possible, of rigid street use. One sees the trackless trolley coming into use, transit facilities are being placed underground or overhead, and traffic, in all directions, is being built up by better ordinances and better regulations. This action is demanded for the benefit of vehicular traffic. It can often be secured by changes in routing and the creation of circumscribing belt lines. Ten years will probably see a great change in this direction.

The economics of transportation has become so well established that it can now be stated that the density of population and the riding habit of the residents of a part of any community determines the kind of transportation which will pay. If a 40-min. headway is indicated, it means that the motor bus must be used. Where it is 20 min., the trackless trolley is the thing. About a 10-min. headway makes the trolley line the advantageous method of transportation. With a 3-min. interval, one must go to high-speed rapid transit. On the basis of such economics, one can very nearly determine the type of system that will work out best, for example, to connect the outlying districts with the down-town section of a large community. The trolleys can go just about so far, and they should be met with trackless trolleys, with buses beyond them.

Housing is inextricably involved with transportation. The two possess psychological as well as social and economic aspects. Certain general principles can be laid down, but the specific case must be handled largely on its merits.

BETTER LIGHTING AS A PART OF BETTER HOMES

By Samuel G. Hibben,* Esq.

The subject of lighting in residences and homes is of interest, partly because so little is known about it and partly because it is such a rapidly growing science.

The scientist or the engineer usually wants to discuss a subject in terms that are concrete or clear, and there are three well known terms used in all discussions of illumination, namely, "candle-power", "foot-candles", and "lumens".

Candle-power is a common term. An illuminant, whether it is a lamp or a larger diffusing unit, will radiate light with a certain pressure, or candle-power. People are not concerned with the "pressure" of radiation of light, only so far as it may cause harm to the eyes. They are concerned, however, with the amount of illumination on a working surface, because if they were to attempt to see a paper on a desk top or a picture on the wall, they could only see it by virtue of the amount of light falling on it, and reflected from the paper or the picture to the eye.

To measure the illumination on a surface, the term "foot-candles" is used. If snow was to fall to the thickness of 1 ft., one would obviously say the depth of the snowfall was 1 ft. Similarly, the thickness of lightfall on a surface is measured in foot-candles. The instrument to measure the lightfall is called a foot-candle meter.

The quantity of light is measured in terms of "lumens" and is the product of the area illuminated and the foot-candles of illumination on that area.

In the application of light, one wishes to know the amount, the direction, and the color. To the physicist, color is merely a matter of wave length, and can be controlled by absorbing certain wave lengths, either in transmission through glassware or reflection from wall surfaces, or a selected light source of a given wave length may be used and, therefore, of color.

In common practice, there is a wide range in color and amount of illumination. The eyes have become accustomed to this variation so that, over short periods of time, one can scarcely notice when there is too much or too little light, unless measuring instruments are used or the conditions are investigated quite thoroughly. For that reason, the subject of illumination in the home needs careful engineering attention, because the eyes of children—adolescent eyes—that may continue to function under insufficient illumination, nevertheless, may be straining all the time, until some serious difficulties result. Conversely, the eyes may be exposed to the rays of bright, glaring lamps. Every one suffers from glare in the same way, but the evil is insidious and affects, particularly, children and elderly persons. The resultant eye troubles are sure to come, although in a slower or milder form. Headaches and similar troubles are caused by eye strain under glare.

Any one individual may safely say that a study of home lighting does not seriously concern him, but with a community, or a group of people, as ex-

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emplified by a large office building, the possible economies or the possible losses must be considered. The economies that a person in his own home might consider to be negligible, will represent, over an entire community, a great saving. One may be astonished to find that in his home or office he can use a light gray or white shade of wall or ceiling and a 40-watt incandescent lamp, and receive the same amount of light on his desk as with a dark yellow or green shade and a 60-watt incandescent lamp. The amount of light reflected from different colored surface varies. Probably 80% of the light is reflected from a good white colored surface, but only 35 or 40% from medium red or tan shades, and perhaps 10% from the dark green.

The average decorator and housewife does not seriously consider the effect of interior colors, wall paper, and finishes on the amount of useful illumination. The kind of paint should be considered, because it has been discovered recently, that paints of the same color, but made from different constituents, will reflect different amounts of light. One can easily demonstrate by mixing lamp black with a white paint, thereby getting the same shade of gray as perhaps from other mixtures, such as green and vermilion. The lamp-black gray reflects less light than the same shade of color made from a green and a red. Lamp black is a heavy absorber of light.

There are many other factors in economic lighting that the home owner may ill afford to overlook. The position of lamps, whether burning tip up or tip down or sidewise, may shorten the life considerably. The proper voltage for Mazda lamps is important. In one modern residence some of the lamps were for 110 volts and some for 115 volts. The question was asked, "What difference does it make? They both burn." The answer is, "If the voltage is 115 and the lamps are for 110 volts, their life will be reduced about one-half. If the voltage is 110 and the lamps are for 115 volts, about 85% of the possible light is obtained."

Another matter in this illuminating problem is that there is probably as much difference between the amount of light absorbed in various glasses as there is in colored paints. A good quality of opalescent glassware may transmit 85% of the light, whereas a poor quality will transmit only 60 per cent. A great mistake is made in most artificial lighting installations in not studying the question of the glassware. A few years ago, the crystal rough frosted glass was used almost exclusively. It was not a good diffuser and was a heavy absorber of light. At present, it is rarely used and then in the cheapest type of lighting fixtures. Its low first cost is lost many times over, through its inefficiency.

The selection of window or sheet glass, used sometimes in skylights and particularly in windows and partitions, is important. The prism window glass has never been seriously studied from the standpoint of lighting, and yet it can be easily demonstrated that a piece of prism or ribbed window glass if turned with the prisms to one side or the other, might change the amount of useful light in a room by 30 or 40 per cent. In other words, if a ribbed glass with the prisms on the inside, or running in the wrong direction, was used in windows where the light comes from without, comparatively little light would be obtained in the rooms. If the prisms are placed on the side

from which the light comes, the illumination might be increased 50 per cent. Many good architects have never thought of this matter; they will place windows or partitions with ribbed or prism glass in them, but the idea of placing the rib one way or the other was never investigated.

In 1920, it was ascertained that of the 30 000 000 or 35 000 000 homes in the United States, only about 6 000 000 were wired for electric light, so that there is a possibility of much improvement in extending the use of electricity for residence lighting. Also, for every 16 or 17 rooms in a house, there is only one convenient outlet, such as a baseboard or plug receptacle. It is surprising that many people in buying, building, or planning homes, forget that the electrical conveniences—portable table lamps, stoves, cooking devices, fans, etc.—must first have an outlet provided for them. In some of the modern apartments it is difficult to find good places to attach electrical conveniences that might be used if the attachment facilities were available.

Take the average residence, for example, in the kitchen one finds far too many installations in which a bare incandescent lamp is used, hanging somewhere over the stove. This arrangement is not only detrimental to good vision, but it is uneconomical. The maximum amount of light from the bare lamp is sent out sidewise and much goes upward, with relatively little projected downward. It is downward, on the stove, that the illumination is wanted, and it would require perhaps 5 or 6 foot-candles on this surface to enable a person to see properly. Yet, from 12 to 20 foot-candles are used on the upper part of the room and practically none on the working surfaces, except what may be reflected from the ceiling or opposite walls on the stove.

The use of combination gas and electric fixtures is poor practice. Too often the electric lamp is too low to illuminate a large area, or points out at an angle that is inefficient. It is better to have the emergency gas-bracket within reach, and a ceiling electric unit controlled by a pull-chain or wall switch.

Many equipments, even in modern homes, use a larger lamp than is intended for the reflector. The lamp bulb extends about half way out of a decorative but inefficient, glass shade. Any lamp that projects out of the reflector is not operating satisfactorily. The ideal method is to have the tip of the lamp bulb about level with the bottom edge of the reflector.

Entrance doorways of the home are quite well lighted by brackets, in some cases the house numbers being painted on the glassware. The usual mistake with such porch brackets, is that they are placed behind opening doors and every time the wind or the children throw the screen door open, the glassware is broken.

In many residences, one still finds the old dome hanging over the library table in service. It is not bad if the lamps are placed far enough in it so that the exposed filaments of the incandescent lamps are not visible. Consequently, if that sort of fixture is used, a screen of silk should be placed under it, or a piece of diffusing cloth, anything that will shade the eyes from the direct view of the light bulb.

The cleaning of lighting devices is important. Glassware, lamps, even walls, all need better attention than they usually receive, and the dust that

will accumulate in a month may reduce the illumination 30% or more. Attention should be called to the fact that either a deeply shielded lamp, a rather dense semi-indirect bowl, or an inverted opaque bowl, are excellent for the bedroom.

Home lighting involves lamps, colors of interiors, cleaning, optical hygiene, amount and direction of light, but most of all, a realization of the vast difference between pleasant livable illumination and just lighting. Lighting and housing conditions can be improved tremendously if a little attention is given to the problem. The manufacturers of lamps, or the larger manufacturers of lighting devices, are ready to furnish service to increase the quality of illumination; and, finally, people should be impressed with the fact that better lighting is not necessarily more expensive.

DISCUSSION

EUGENE W. STERN,* M. AM. SOC. C. E.—One phase of the housing problem —and a very important one—has not been touched on, that is, the question of financing. It has been stated that the way to solve the housing shortage is to build more houses; but one of the present difficulties is the impossibilty of getting money at such reasonable rates of interest, as will enable the builder to construct and compete in price with the houses built under more favorable conditions.

The gross cost of a building is made up of a number of items, one of which is the actual builder's cost. Another, but a most important one, although not often thought about in this connection, is the cost of financing the building operation from its very beginning to its completion, and it is surprising what a large item this becomes in the total cost of construction.

In New York City, and in other cities of large population, the building of homes or tenements has been undertaken in the past almost entirely by professional builders who have specialized in this class of construction. They are the ones who have constructed those huge rows of houses, likewise the vast majority of tenements and apartment houses, which one sees in all large cities. They are in business to make money on their ventures, as they should. Private individuals who build their own homes are such a negligible factor in the housing situation that they need not be considered. The speculative builder of low-priced tenements suitable for working people has practically ceased operations, and one of the reasons is the fact that he cannot borrow money at reasonable interest. Money rates are high, and it seems likely that these high rates will prevail for a long time. As long as the Governments of the world, particularly European Government, are willing to pay from 8 to 10% for money loans in the United States, it cannot be expected that builders can get money at 6 per cent. The speaker is informed on reliable authority that the cost to them is anywhere between 10 and 15 per cent.

The speaker believes that here is a situation in which the State can help very materially, namely, by assisting in financing housing projects. It has been done in France for more than half a century by an organization semi-governmental and semi-private, whereby long-term loans extending to 20 or 30 years, amortizing annually and on reasonable rates of interest, are made to people who desire to build.

Although the high cost of building is partly due to combinations among material men and manufacturers to boost prices artifically and is another factor in creating high rents, in the last analysis, one of the most important, if not the most important, of all the factors which enter into this complicated problem to-day, the speaker believes, is the inability to finance housing construction on reasonable terms.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—Attention has been called to the relative cost of labor and material, which is generally assumed to be about "fifty-fifty" or "forty-sixty". Those are about the proportions for work done

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in New York City, but the 50% allowed for cement, steel, and the other materials used, means that the contractor who produces such materials has already put in about "fifty-fifty" for labor and material; therefore, as a matter of fact, the total amount actually paid to labor, if traced back to the mine, factory, cement works, etc., would be a much larger percentage. It would be interesting to know what the percentage really is that goes to labor.

Mr. Purdy referred to the fact that the pollution of the soil is no longer permitted in Lower New York, but pollution has existed so long, with possibly unknown leakages still going on, that New York quicksand has received a disagreeable odor. If one will walk up to the top floor of an old building in the lower part of Manhattan, about three or four o'clock in the morning, he will probably detect this disagreeable odor.

The discussion so far has been principally on how to rectify the present deplorable conditions by zoning. There is another feature easier in some respects, and much harder in others, that is, what restrictions for regulating the height and size of buildings, etc., should be made in laying out a new city. In laying out the towns of Pullman, Ill., and Gary, Ind., it was possible to obtain all the space needed, and the purpose for which these two towns were built, was more or less restricted.

Nelson P. Lewis,* M. Am. Soc. C. E.—It has often been said that congestion does not necessarily mean the number of people to the square mile, or to the acre, but that it is indicated by the number of people in a room, and just as bad housing conditions can be found in sparsely settled communities as in great cities—sometimes even worse—although in the former case such conditions can be more readily corrected.

It has been stated that it is difficult to induce people in suburban districts to buy and retain lots of a size greater than barely to accommodate a house and a garage; this statement is significant and somewhat disturbing.

The suggestions given for apartment house design, and the statement made as to the satisfactory results to both owner and tenant, are quite encouraging. The question naturally suggesting itself is, "Cannot some policy be adopted that would encourage less intensive use of ground area in both apartment house and detached house districts?" The assessing and taxing officials might endeavor to place a premium on more generous allowance of land in housing enterprises by fixing the assessed value of the land, for purposes of taxation, in accordance with the area occupied. Assessments appear now to be fixed according to the potential value of the land, provided it can be covered to the utmost permissible limit. Suppose, for instance, assessing officials, who are usually allowed great latitude, should fix, for each locality, a unit land value, which might be different for every block, and wherever plots have been built on they should fix, as the taxable value, such percentage of that unit value as would be determined by the percentage of the area of the lot built on. For instance, it has been pointed out that the old type of apartment house covered 79% of a plot 100 ft. square and that the proposed new type covers only 62 per cent. For purposes of illustration, assume that where buildings cover 80%

* Cons. Engr., New York City.

of the lot area, the land should be assessed at its full unit value already established, and that where less than 80% is covered, the value should be fixed at a figure inversely proportioned to the proportion of the lot occupied. For instance, if 62% only is covered, this represents 77½% of 80%, and the assessed land value on a plot so improved, therefore, might be fixed at 77½% of the normal value for this particular district. On the other hand, if 90% of the plot was covered, this 90% being 112½% of 80%, the assessed land value, in this case, might be fixed at a value 12½% greater than the unit value already established. Possibly constitutional objections would be found to the formal adoption of such a policy, especially if it were to be given legal sanction, but it has already been pointed out that assessing officials have much discretion, and it might be well to try out such a policy, the legality of which might have to stand the test of Court proceedings.

Westchester County, in New York State, has been, as far as the speaker knows, the first county, at least in this part of the United States, to establish a County Planning Commission. Probably the Commission has not yet attempted to exercise much authority, but it is a hopeful sign. If counties contiguous to great cities do not themselves attempt to control development, it will be necessary, in the public interest, to give the cities themselves some degree of jurisdiction over the planning of territory beyond their corporate limits. This has already been done in many cases, and a canvass of 27 typical cities made by the speaker shows that in 10 of them the City Planning officials have jurisdiction varying from 1 mile to 15 miles beyond the city boundary. In the case of the other 17, which included both New York City and Philadelphia, there is no jurisdiction whatever outside the city limits.

Mr. Wagner has drawn attention to the desirability of excluding residences from districts devoted to heavy industry. This appears to the speaker a perfectly logical thing to do. If such industries are forbidden to invade districts set apart for human habitation, because their presence will be injurious to health and comfort, it is just as logical to say that buildings for human habitation shall not be erected in places set apart for such industries.

JOHN IHLDER,* ESQ.—If the plans (Fig. 4), presented by Mr. Thomas do what he claims for them, they are revolutionary. In housing, the great fight has been against constantly greater and greater land overcrowding. The evil effects of land overcrowding as applied to considerable areas and over long terms of years can be demonstrated, and because of this the zoning ordinances were secured. The individual lot owner, however, still believes that the more building he can crowd on his lot, the more profit he will make, because he believes that this crowding gives him more rentable floor space. Of course, he admits that if his neighbors follow his example, all of them will lose; but he thinks of his neighbors' lots only as sources of light and air for his own building. If Mr. Thomas can prove that as much rentable floor space and, consequently, as large a revenue may be secured in a building occupying a smaller percentage of the lot, he has brought even immediate pecuniary advantage to the side of better housing. Continued or long-time pecuniary advantage has always been on the side of better housing.

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Mr. Leavitt has stated that he has found an increasing tendency to crowd the land. The speaker accepts Mr. Leavitt's instances, but not his conclusions. The tendency to crowd and overcrowd the land has always existed, but, although it still persists, it has been noticeably checked, not only by zoning ordinances which are being adopted all over the country, but by an informed opinion on the part of individual owners. Great estates of many acres may not be as popular now as they were some years ago, high taxes and reduced incomes have a contracting effect. Well-to-do people may be building smaller houses on small grounds, along the edges of private golf courses, and finding their opportunity for expansiveness on the golf course they own in common. This may be an indication of the growth of a community spirit that is interesting enough to warrant some study.

As for the placing of tight little rows or groups of houses among the more open buildings of New York and Philadelphia suburbs, this has seemed to some of those who have watched it to be a manifestation of the old desire to get as much as possible for as little as possible. These tight little groups, like the occasional apartment houses set in residential suburbs, owe much of their appeal to the more generous standards of their neighborhoods. Their occupants profit by what others pay for. So, of course, a considerable number of them will sell readily and to the profit of the land speculator. Their presence, however, immediately decreases the desirability of their neighborhood, and as their number increases, their appeal decreases. This is only the repetition of an old experience through which many now closely built sections of American cities have gone.

Mr. Lewis stated there are just as bad housing conditions in sparsely settled areas of New York City as there are in the greatly crowded areas. To that the speaker takes decided exception. That there are intolerably bad housing conditions in sparsely settled areas he admits; but they differ from those of the crowded tenement districts of Manhattan in that they are superficial and easily removed. They are largely insanitary conditions, and once the public has made up its mind that it will be clean, that it will pay for water, sewers and garbage collection, they will be removed. Small-house cities like Philadelphia, for example, may, if they will, make themselves very good cities from the housing point of view; but when land has become overcrowded with tall barrack tenement houses, as it has in Manhattan, the evil is beyond any practical remedy. The difference is one not of degree, but of kind. Land overcrowding is the greatest and the most nearly incurable of housing ills; consequently, there cannot be as bad housing in sparsely settled areas as there is in crowded New York City.

Henry H. Curran,* Esq.—The speaker wishes to emphasize the importance of better co-operation between those who plan for cities and those who, as city officials, carry out the plans.

Edward S. Rankin,† M. Am. Soc. C. E.—It may be of interest to cite an instance which occurred in Newark, N. J., during the housing shortage,

* New York City.

† Engr. in Chg., Bureau of Sewers, Newark, N. J.

along the line of Mr. Lewis' statement that no houses should be permitted in an industrial district. One of the largest manufacturing companies of Newark owned a tract of land bordering the meadows within what is known as the heavy industrial zone. The company proposed to build on this tract a large number of houses and brought all possible arguments to bear on the officials to allow this to be done. The tract was within reasonable distance of the plant, which was situated on the meadows, and the company proposed to sell these houses, or rent them at a reasonable cost, to its employees. After due consideration, however, the project was rejected because it did not conform to the zoning ordinance.

BENJAMIN A. HOWES,* M. AM. SOC. C. E.—Water supplies in practice are of such varying chemical content that no set of rules as to materials recommended for, and life of, piping can be made universal. In the case of iron and steel pipe, one water appears to form a slight but protective coating in the pipe, whereas, another forms a growing coating which finally clogs the pipe, and a third attacks the pipe and eats through it.

Chemical treatment of water to reduce its bacterial or mineral content is becoming more general. Such treatment may increase or reduce the virulence of the water's attack on the piping. Thus, the decision as to what kind of piping is economical for houses is dependent on the character of the particular water to be used.

In the past, lead pipe was used extensively for house distribution where the water attacked iron pipe rapidly. The speaker has never known of lead poisoning resulting therefrom, because the water appears to form an insoluble coating on the inside of the lead pipe. However, the use of lead pipe in houses is seldom justified: First, because the pipe must be continuously supported to prevent sagging and consequent strain and water traps; and, second, because rats will gnaw through lead pipe for water when cut off from other safe sources. The speaker has seen instances where extensive damage to plastering and decorations resulted from rats attacking the lead distributing pipes. In another instance, the rats gnawed a large hole in the 4-in., heavy lead bend connecting the toilet bowl to the soil line, rather than go to the open flush tanks or toilet bowls in the rooms, or depend on rain water in the open.

Some iron and steel pipes give long service (more than thirty years), especially when galvanized. Again, wrought-iron and steel pipes on the same waters rust or clog with rust in a few years. Hot water is usually much more virulent than cold water in its attack. Such attack has been met in cases where renewal of the pipe would occasion great expense by passing the hot water and, if desirable, the cold water through storage tanks in which many sheets of thin iron or steel plates, such as metal lath, were placed to exhaust the active contents of the water before it enters the piping system. These plates were renewed from time to time and the deposited rust was removed. Rusting or corrosion seems to require, among other things, the presence of active or free oxygen. The cure in this case seems to have resulted from the removal of the oxygen in the form of rust (iron oxide).

* Cons. Engr., New York City.

The life of iron or steel pipe appears to depend not so much on whether it is wrought iron or steel, but on the process and completeness of manufacture. If the material is worked during manufacture into a uniform homogeneous material, it will have long life. If it is rapidly produced and carelessly worked, so that there are hard and soft spots, areas of varying carbon content, minute segregation of impurities, and the physically different cooling products of iron, there will be adjacent areas in contact, that will have different electrical potentials when in contact with natural waters, and corrosion will be prompted and the life of the pipe reduced.

Well-made brass pipe, in the speaker's experience, has been durable for both hot and cold water. Probably this is because it is so completely homogeneous. Where it is in contact, in the water, with iron or steel, such as bolts, screws, washers, and faucet fittings, there is rapid corrosion of the iron, probably by electrolytic action. The small difference, however, in present installed cost, quoted by Mr. Ham, indicates that brass pipe should be generally used for distribution of water within the house. Sometimes, there is complaint that water from such pipes has a "brassy" taste. This has been met by using "tinned" pipe. The appearance is improved, and the plumber is likely to be more careful in his threading and making up, if the pipe is also tinned outside. Economy is advanced with no apparent loss of long life if heavy, galvanized, malleable iron fittings are used instead of brass fittings, at least for the cold-water runs. Before the World War, brass pipe tinned both inside and out, cost about $5 more per 100 ft. than untinned pipe.

In the case of some New England waters which have a bad reputation for rapid attack of iron and steel pipes, very good life is obtained for that part from the street main to the cellar, by using iron or steel pipe lined with from $\frac{1}{16}$ to $\frac{1}{8}$ in. of neat cement (Portland or natural). This lined pipe is a staple article in New England.

D. L. Turner,* M. Am. Soc. C. E.—In the speaker's opinion, one of the most important phases of the housing problem—if not the most important one—particularly in a great city, is transportation. Engineers think too much of the house alone, that is, of the home alone. As workers, all Americans recognize that with the home there must also be the workshop; it is essential that the two be connected. In any housing proposition, it is essential that transportation be considered; and yet the question of transportation is seldom considered except in so far as the private investor is willing to provide it.

A home may be provided with water, sewers, and light, but unless it is connected with the workshop, it is useless to those who occupy it. The transportation phase of the problem is as much a social problem, and is just as important, as any other feature of the housing problem.

F. W. Look,† Jun. Am. Soc. C. E.—When the zoning ordinance of Paterson, N J., became a law backed by the city police power, property owners, adversely affected by the zoning law, objected in that their property would be depreciated by the fact that only two-family houses would be permitted next to an

* Cons. Engr., Transit Comm., New York City.

† Paterson, N. J.

existing store. That particular residential plot of land has dropped in value, compared with the recent advance of the adjoining business property. Other property owners object to the building regulations governing Class A Residential Zones.

The value of narrow, odd-shaped lots in this high-class zone, drops to 33% of the larger improved lots. Some land-owners claim unjust treatment by the prohibition of buildings in that locality permitted in Classes B and C Residential Zones.

A Board of Appeals has been instigated to regulate these and similar cases. The question, therefore, arises as to how this problem will work out; to what extent a few must sacrifice their small gains for the benefit of the entire city.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).—One of the most important influences affecting the National Housing Problem is the modern zoning ordinance. Zoning, however, is closely related to the park and boulevard plan and this, in turn, to the street system. The street plan is dependent on the railroads and often on seaport requirements. Furthermore, there can be no adequate design of streets and transportation lines without regard to the location of industries and commerce. The problem of trade and manufacturing is, of course, directly dependent on the labor problem. To reduce labor "turnover" by properly housing the workmen is one of the chief ends of the city plan. Thus, passing around the circle, the starting point has been reached again, namely, the housing problem, which, in its relation to zoning, is the subject of the following discussion.

The chief characteristics of modern city planning are unity and comprehensiveness. The elements of the plan must be harmonized; no feature can be altered without affecting others and thus affecting the whole plan. The writer finds it difficult to limit himself to the influence of zoning on the housing problem for the reasons already given and also because of the fact that adequate housing is only one of the benefits conferred by zoning. The other benefits re-act powerfully on the housing problem.

To define the policy and practice of zoning is impossible because of the rapidity of its development. It would be folly to limit its scope or possibilities. Those who have made a study of it during the few years of its operation are unanimous in their confidence in its potentiality. Zoning, or districting, consists in the dividing of a city into districts for the regulation and betterment of living conditions, of business, and of manufacturing. Its aim is efficiency —to divide the city's territory so that it may be put to its highest and most specialized use. Its object is the segregation of homes, factories, and business houses, and the establishment of a limited control over all building operations as far as the height, size, and use of the buildings are concerned. Zoning is the constitutional use of the police power in the interests of health, safety, and general welfare. Zoning is not merely a prohibitory ordinance, a protection for dwellings and a restriction for manufactories and business houses, but rather a great constructive power for the up-building of the whole city.

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In urban life, the first restrictions on property were the fire laws which were necessary for the mutual safety of families living closely together. No one would now question the desirability of fire laws, and yet they were what might have been called a violation of private property rights. Next, came the building laws which were enacted in the interest of the public health and safety, with their minute regulations concerning building materials, sanitation, size, and management of buildings, clearly a violation of the old-time idea of the sanctity of property rights. Next, came restrictions on the height of buildings, also in the interest of health and safety. Restrictions were also being placed on manufacturing of a nature prejudicial to adjoining property. Lastly, has come the full-fledged modern zoning, governing the shape and size of all buildings and the use to which they are placed, clearly also a violation of property rights, as formerly understood, and yet being sustained by the State Courts and the United States Supreme Court, not only in regulations governing future operations, but applying retroactively to those of the past as well.

Zoning stabilizes property values by eliminating speculation. Property values increase to a certain point or limit of intensiveness of use. Beyond this, congestion results; a reaction sets in and values decline. Zoning prevents inflation of values. Zoning protects the public-spirited house owner who sets his house back from the street. Heretofore, an owner has been impelled to build to the street line for his own protection. If he did not build to the sidewalk, his neighbors would, thus pocketing his home between two projecting buildings. Many a guileless owner, trusting to the comity of his neighbors, has thus been deprived of the value of his property.

Zoning protects the home. It provides suitable home sites for both rich and poor and depopulates the slums. By preventing congestion in tenements, it promotes morality and with its requirements providing for light and air, it promotes sanitation. Zoning encourages home building and, therefore, tends to reduce the tidal movements of the city's population. No "own your own home movement" can be successfully carried through without it.

At the present time, the art of zoning is taking up advance positions. Many of its well established principles have not been tested in the Courts, although the judiciary, whenever appealed to, has quite regularly given decisions supporting the zoning laws whenever they can be shown to be based on requirements of health, morals, and public welfare. The United States Supreme Court has even gone so far as to approve of the ejection of a brick yard from the residential section of Los Angeles, Calif., giving its opinion that "a brick yard has a clear physical effect on surrounding property". However, support of this character is not invoked in the more recent ordinances, although their legality is strengthened by it.

Zoning is not so much a matter of theorizing as it is a matter of diagnosis. As an eminent engineer has said, it is four-fifths a matter of fact and one-fifth a matter of skill and judgment. Its aim is to avoid congestion. While encouraging the building of private homes, it must offer no obstruction to the building of apartment houses. It has gone on record most emphatically, however, as being opposed to congestion in tenements and to what has been called the "warehousing" of human beings. Although districting discourages certain

kinds of objectionable business, it is most helpful to others of a legitimate character. Thus, by assuring prospective home builders of the permanence of the residential neighborhood in which they would build, it stimulates house construction.

In modern zoning, the town is divided into three classes of districts: First, the height districts; second, the area districts; and third, the "use" districts. Regulation is accomplished by the exercise of the police power without compensation. Districting is not concerned with the requirements of the building laws, however important they may be in matters of safety and sanitation; it deals with the larger generalities.

The height districts are determined by the broad principle of equal rights to air and light, or, as the English say, by the "ancient lights". Districting with regard to height is concerned principally with office buildings. Apartment houses must also be controlled. In cities without a zoning ordinance, apartment houses have been built so high that they cut off light and air from surrounding houses the value of which may drop as much as 30 per cent. There are two methods of regulating the height of buildings: First, by a flat limit; and, second, by specifying the allowed height as a function of the street width. The latter method is the more popular and has the merit of encouraging street widening, since the wider the street, the higher the building may be.

It has been said that "the problem of the last generation was the supply of gas and water; the problem of the next is to provide light and air". Merely limiting the height of buildings is not enough, for a building may cover the entire lot and, therefore, cut off light and air from windows adjacent. The object of area districting, however, is not only to provide light and air, but also to prevent congestion, although, in this respect, it has not been very effective. The zoning ordinance of Newark, N. J., limits the population directly on the area basis. In the tenement district in New York City, in 1914, the average density was 852 persons per acre. The Newark ordinance is unique in character, having been the first effective attempt of its kind, in the United States, to prevent land crowding. By the Newark ordinance, the maximum population is 80 per acre, only about one-tenth of the New York City density.

Zoning on the basis of the use to which buildings are placed, has been sustained by the Courts of several States and by the United States Supreme Court. The most up-to-date practice of zoning calls for four "use" districts, the residential, the manufacturing, the business, and the unrestricted or nuisance.

"A city is something more than a place to work in", says H. S. Swan, "it is also a place where people live, and although people must work to live, they must live to work". Living conditions will rightfully be considered first. The residential district is sometimes divided into two sub-districts, one exclusively for single-family homes and the other for single-family homes and apartment houses. Since in some cities there is doubt as to the legality of legislating on the basis of the number of families in a house, the requirement is made that the houses in the exclusively residential district may not have more than two

stories, thus effectively legislating against tenement conditions. Owing to the evident need for apartments by small families and by those who are not in a position to buy or rent a single-family home, apartment houses must be recognized and allowed. Their construction, of course, must conform to the building code and their height and area to the zoning requirements. The apartment house districts are near the business district and are determined by apartment houses already built. In the same district, single-family dwellings must be permitted, although the district will become more undesirable for such dwellings and more desirable for apartments.

The housing problem must go along with the transportation problem. Without proper transportation, houses will be huddled together, especially workingmen's homes, thus resulting in abominable social conditions. With adequate transportation provided by the city plan, the residential zone can extend to great distances from the centers of industry and business, thereby securing the advantages of air, light, and quiet. In planning the residential district, consideration must also be given to the wishes of those who prefer to live near the scenes of activity. Many workingmen prefer so to locate, and their wishes should be respected. At the same time, the contrary principle must be borne in mind by the planner, that owners and renters do not know what is best for them, that quite frequently people of small means live on expensive land and people who are wealthy live on land that is cheap. It would be far better, in many respects, if working people could be induced to live at a distance from their places of business, providing that adequate facilities could be provided for their transportation. This principle is denominated "decentralization", and is the cardinal idea in the building of what are called, "garden cities". Another method of dispersing the residential population is by inducing the factories to remove to the outlying regions. Heavy industries such as iron founding and steel manufacture are often willing so to remove; but even with this accomplished, there is still the danger of slums and congestion about the new center.

The manufacturing district is located with respect to railroad, wharfage, power, and labor facilities. Of these requirements, the labor problem is one of the most important. To reduce labor turnover is one of the manufacturer's chief concerns. The municipality by providing him with facilities for obtaining and retaining satisfactory labor is rendering him a great service. The city is in a position to perform this service by passing suitable zoning ordinances to protect the homes of his employees and to provide them with means of transit.

The business district is located in the heart of the town with radiating streamers extending out along the principal highways. In it, the interests of business must be conserved by prohibiting manufacturing and objectionable trades. Some authorities would also prohibit residences.

The unrestricted or nuisance district accommodates industries not permitted in the manufacturing district. There is, however, a serious objection to making any district unrestricted, owing to the tendency of human beings to live there, under conditions prejudicial to health and safety.

Using the word, "district" in an entirely different sense, there are also the so-called "blighted districts". Blighting is the dread of real estate operators and investors and is the direct result of unregulated building. A factory or garage may intrude into a residential district and ruin values in all directions; so, also, may an apartment house, constructed in a high-class residential district. Zoning is as good a protection as life or fire insurance and possesses the great advantage that it costs practically nothing. Then, too, there are improvements which produce blighting, "depreciatory improvements", as they are called. Thus, a surface or elevated car line will quickly blight a residential district, a municipal gas plant, a business district, or a change of railway location, an industrial district. To prevent "blighting" is one of the chief objects of zoning.

William H. Ham,* M. Am. Soc. C. E.—For the complete plumbing of seven types of houses, the speaker obtained cost figures which varied from $408 to $475 per home, using 1-in. steel pipe for the water supply. The use of wrought-iron pipe would have added about $8 to the cost, and brass pipe from $35 to $38. The cost of piping is a small part of the cost of the house. These houses averaged in value from $4 000 to $8 000.

The selection of the proper kind of pipe is a serious question in all communities. Data on the carrying capacity of new to four-year old brass pipe in comparison with the capacity of steel or iron pipe of the same age would be desirable to people who are buying homes. The speaker is removing a certain kind of pipe in thirty-nine apartments, in which there were 120 leaks in the hot-water system alone, and about the same number in the cold-water system. The 1½-in. pipe is being replaced by 1-in. pipe, and the 1¼-in. pipe is being replaced by ¾-in. brass pipe. The speaker did not know, before he investigated the subject, that for a $5 000 or $6 000 house the difference in the cost for the use of brass pipe from the street to the house, and out to the hose connection, was only $38. Such matters should be brought to the notice of the public as they are important facts which affect the housing situation. The speaker's company bought from the Government 257 houses in which steel pipe was installed, which will have to be removed in about 6 years. After 2½ years of use, some of the pipe is rusted to such an extent that it is being removed in the damaged places.

It would be well if water supply engineers not only codified these facts, so that instead of using 1¼-in. steel pipe, one might use ⅞ or ¾-in. brass pipe, but also furnished data as to localities in which brass pipe may be safely used.

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THE DESIGN OF AERATION UNITS AND SEDIMENTATION TANKS FOR THE ACTIVATED SLUDGE SEWAGE DISPOSAL PLANT AT MILWAUKEE, WISCONSIN

By Darwin W. Townsend,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Harrison P. Eddy and Arthur L. Shaw.

Synopsis

During the last seven years the activated sludge process of sewage disposal has been developed from miniature laboratory models in the first stages of scientific observation to a practical and efficient working process capable of purifying sewage to a high degree.

The development of this process has claimed the attention of sanitarians throughout the civilized world, and promises to continue doing so in the future.

The municipal authorities of many towns and cities in the United States and Europe have investigated and adopted the process as most applicable to the conditions involved in their problems of sewage disposal.

The City of Milwaukee, situated on the western shore of Lake Michigan, and at the junction of the Milwaukee, Menomonee, and Kinnickinnic Rivers, which flow through the city from the north, west, and south, respectively, realized a great many years ago the necessity of preventing the sewage-laden rivers and inner harbor from discharging their ever-increasing filth into Lake Michigan, the source of the city's water supply, and on the bathing beaches on the Lake shore.

In 1914, the then recently created Sewerage Commission established a sewage testing station on Jones Island, at the outlet of the Menomonee Intercepting Sewer, and provided facilities for determining the applicability of

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the various universally recognized processes of sewage disposal, to the treatment of Milwaukee sewage.

During the latter part of 1915, the activated sludge process, then recently introduced in England, was given its first trial in tanks constructed at the testing station for this purpose. The results obtained with these tanks and with other equipment subsequently installed, were encouraging, and during the following two years sufficient progress had been made in the development to warrant the Sewerage Commission in adopting the process for the treatment of the sewage of Milwaukee.

In comparison with the older and better known processes of sewage treatment, the activated sludge process demonstrated its superiority in many ways, the most important of which are:

1.—Rate of treatment per unit of area.
2.—Degree of clarification.
3.—Degree of bacteria removal.
4.—Commerical value of the sludge.

This paper deals only with the particular phase of the design implied in the title, namely, the aeration units and sedimentation tanks, the designs for which are fully developed. The designs for the fine screening and sludge de-watering plants are not fully developed, and, therefore, are referred to only casually by the writer.

That the sludge can be successfully de-watered in filter presses was demonstrated at the testing station during 1919 and 1920. Throughout this period, extending from the fall of 1919 to the fall of 1920, two types of filter presses—the leaf type and the plate type—were used almost continuously, and the sludge was de-watered to a suitable consistency for handling in connection with further moisture reduction in rotary dryers.

In the preparation of this paper an attempt was made to select from the voluminous supply of notes, data, and reports in the writer's files, only matter considered pertinent to the decisions made to date relative to the final design of the aeration units and sedimentation tanks for the activated sludge sewage disposal plant of Milwaukee.

A number of comparatively new terms in sewage disposal literature, applicable, probably, only to this particular process of sewage treatment, will be explained as fully as possible.

The writer is entrusted with the development of designs for the Milwaukee Sewerage Commission, and presents this paper, with the hope that it may contribute something to the art of sewage disposal and help at some future time in the solution of problems similar to those on which engineers have been engaged in Milwaukee.

General Arrangement

The visualized conditions pertaining to the operation of a large activated sludge disposal plant suggested an arrangement of units similar to that of a large mechanical water filtration plant wherein convenience of operation and completeness of control are paramount.

The general idea followed in laying the foundation for the design of the plant at Milwaukee was to secure an arrangement which would provide operating galleries immediately adjacent to the rows of aeration and settling tanks and place in them all the necessary appurtenances for the complete control of the air, sewage, and sludge passing through the plant.

Referring to Plate VI which shows the entire layout, the plant will be seen to consist of twenty-four aeration tanks and fifteen settling tanks—one row of aeration tanks and one row of settling tanks on each side of the east and west center line of the plant.

The sewage, after passing through the grit-chambers and fine screens, and after receiving the proper quantity of activated sludge at the outlet end of the fine screens, will pass through the mixing channel between the fine screens and the plant and will enter the main feed channel at the extreme western end of the plant. At this point, the flow will divide, half of it going through the two feed channels which supply the aeration tanks north of the plant center line, and the other half going to supply the aeration tanks south of that line. Each aeration tank is separated into two compartments by a baffle-wall, thereby causing a reversed flow. The flow enters the tanks through pipes in the end walls of the west compartments and leaves through pipes equipped with meters at the end of the east compartments. The outlet pipes pass through the two feed channels and discharge into the mixed liquor channels which surround each of the settling tanks on three sides.

The settling tanks take their supply from the mixed liquor channels through submerged gates in the east and west walls of each tank, the effluent from each tank being collected in troughs which discharge into each of the main effluent channels running east and terminating at Lake Michigan. The sludge is withdrawn from the bottom of each settling tank and discharges into the two return sludge conduits which convey it to the return sludge pumps in the northeast corner of the main power house. From these pumps, it is forced under pressure to the point of feed at the fine screen outlet, as previously mentioned.

The plant is calculated to provide ample treatment facilities for an estimated population of 588 750.

Aeration Units: Rate of Treatment

Results obtained from the operation of the demonstration plant at the testing station indicate the practicability of operating the aeration tanks at rates as high as 20 000 000 gal. per acre per day without falling below the adopted standards for the effluent. It was decided, however, to adopt 15 000 000 gal. per acre per day as the conservative rate on which to base the design.

The stated rate of 15 000 000 gal. per acre per day applies to the quantity of sewage treated on each net acre of horizontal sewage surface in the aeration tanks only, the settling tanks, channels, and walls not being included in the computed area. When treating sewage at the rate of 15 000 000 gal. per acre per day, in a tank with a 15-ft. depth of liquor and with 20% by volume of activated sludge in the mixture, the corresponding period of detention closely approximates 6 hours.

In computing time of detention, displacement is assumed not to occur in that part of the tank occupied by the sludge. In other words, if a sludge content of 20% is maintained in the sewage, the volume represented by this percentage of sludge is assumed to occupy space in the tanks permanently and is not available for detention purposes.

Each of the twenty-four aeration units is 236 ft. long, 44 ft. wide, and 15 ft. deep from the liquid surface to the top of the diffuser plates, and based on a rate of 15 000 000 gal. per acre per day, with a sludge content of 20%, will treat approximately 3 580 500 gal. per day.

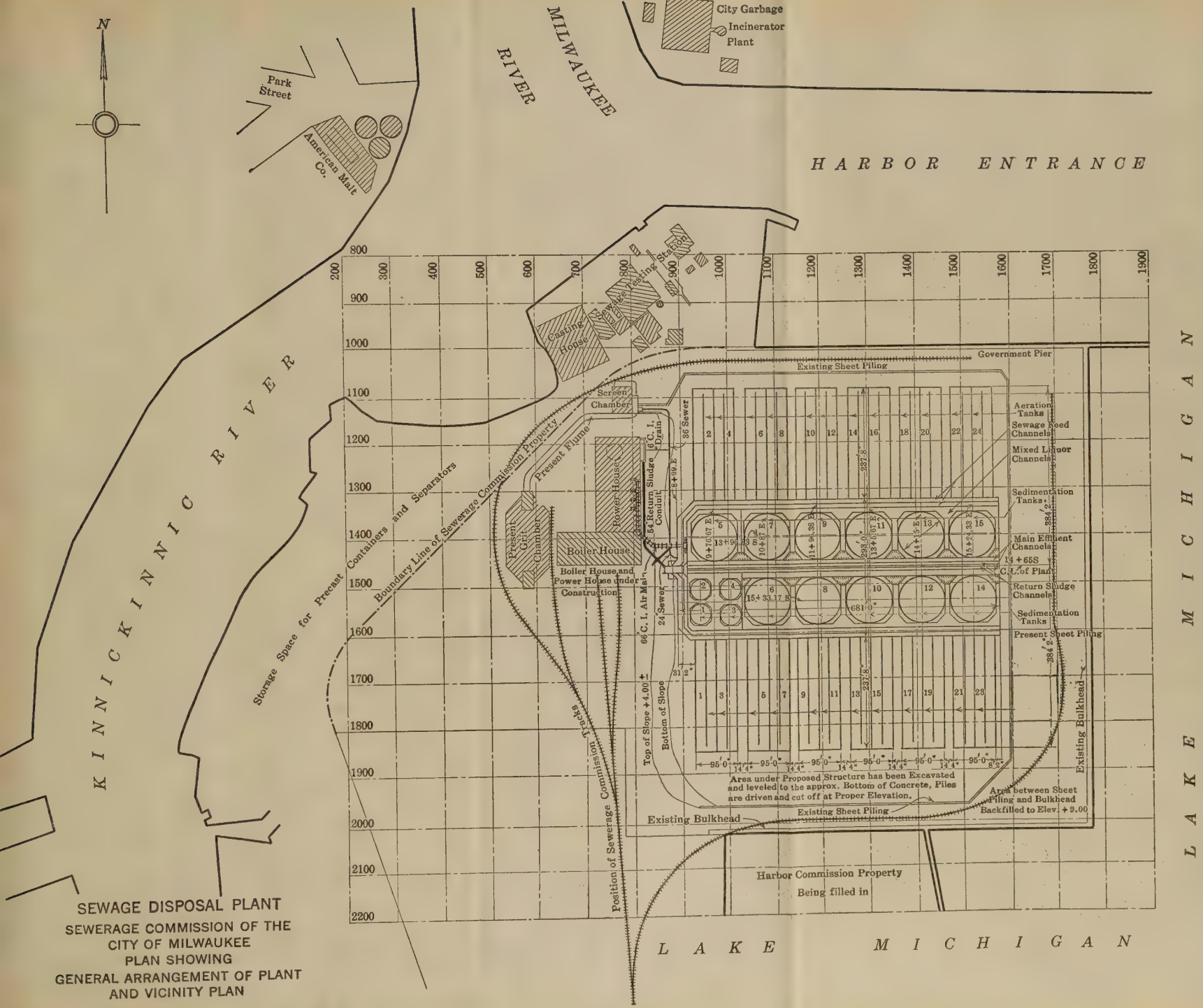
The volume of each tank is about 156 000 cu. ft., of which 31 200 cu. ft. are reserved for the 20% of returned sludge, leaving 124 800 cu. ft. of space available for the detention of the sewage. The detention period corresponding to 3 580 500 gal. per day passing through a tank the volume of which is 124 800 cu. ft., will be, therefore, about 6 hours and 17 min.

TABLE 1.—Estimated Average and Maximum Rates of Sewage Flow per Day for Disposal Plant Design.

(From 1915 Annual Report.)

Year.	Average flow, in gallons per day.	Average flow, in gallons per day, with storm water.	Maximum flow, in gallons per day.	Maximum flow, in gallons per day, with storm water.
1914.....	50 283 000	51 583 000	75 400 000	113 210 000
1930.....	85 183 000	86 483 000	127 800 000	165 610 000
1950.....	128 775 000	130 075 000	193 200 000	231 010 000

In an aeration tank, the time of detention will vary with the rate of sewage flow and the percentage of sludge maintained in the plant. If sludge is returned at a constant rate, the sewage flow will be the only variable which, of course, will fluctuate through rather wide ranges, as shown in Table 1. However, if the sludge is returned at a rate corresponding to the fluctuating sewage flow, the detention period, based on an average rate of flow, would be more difficult to determine, owing to the increase or decrease in the flow intensity of both sewage and sludge during a 6-hour period. As an illustration, the sludge and sewage may be entering a tank at a rate corresponding to the sewage flow for the particular time under consideration, and, at the same time, the sewage and sludge leaving the tank would correspond to the flow which occurred 6 hours previously. Returning sludge at a fluctuating rate would require automatic pumping equipment synchronized with the sewage flow. Such equipment does not appear to be warranted on account of the fact that returning sludge in this manner contributes nothing to the efficiency of the plant. Owing to the "lag" through the plant, due to the large capacity of the tanks, it is questionable whether the synchronous method of operation could be controlled so as to approximate even the conditions of operation which, at first thought, appear to be the most desirable. As inferred previously, the Milwaukee plant is being designed to operate on a constant supply of returned sludge. This is based on the theory, which has been demonstrated successfully in the testing station plant, that the constant supply of sludge carried, is aerated



SEWAGE DISPOSAL PLANT
SEWERAGE COMMISSION OF THE CITY OF MILWAUKEE
PLAN SHOWING
GENERAL ARRANGEMENT OF PLANT AND VICINITY PLAN

sufficiently during the low night flows, so that the plant will treat successfully the peak flow of the day without becoming impoverished in the effort. In other words, the designs are based on a minimum constant supply of well aerated sludge, rather than on a larger and variable quantity aerated probably to a lesser degree of activity.

Each aeration unit is designed for a reversed or two-way flow, and the total length of travel is about 475 ft. The sewage which has been previously mixed with the sludge in the feed channels, enters each tank through two 30-in., gate-controlled, inlet pipes in the end wall and below the sewage surface and leaves the tank through a 24-in. outlet pipe equipped with a Venturi meter. The outlet pipe discharges into the mixed liquor channels which supply the settling tanks. The liquid level in the tanks is maintained practically constant by an overflow weir 12 ft. long, which forms one side of an outlet box connecting directly with the outlet pipe.

Figs. 1 and 2 show the general features of an aeration unit. The ratio of square feet of diffuser area to square feet of horizontal liquid surface is approximately 1 to 4. Each of the cross-containers holds nine diffuser plates and each of the containers running lengthwise through the centers of the two compartments in each unit, holds seven plates. The containers running lengthwise form a gutter in each compartment, to be used for drainage purposes when emptying a tank. Seven plates are used in each of these containers for the purpose of facilitating the standardization of the air-piping.

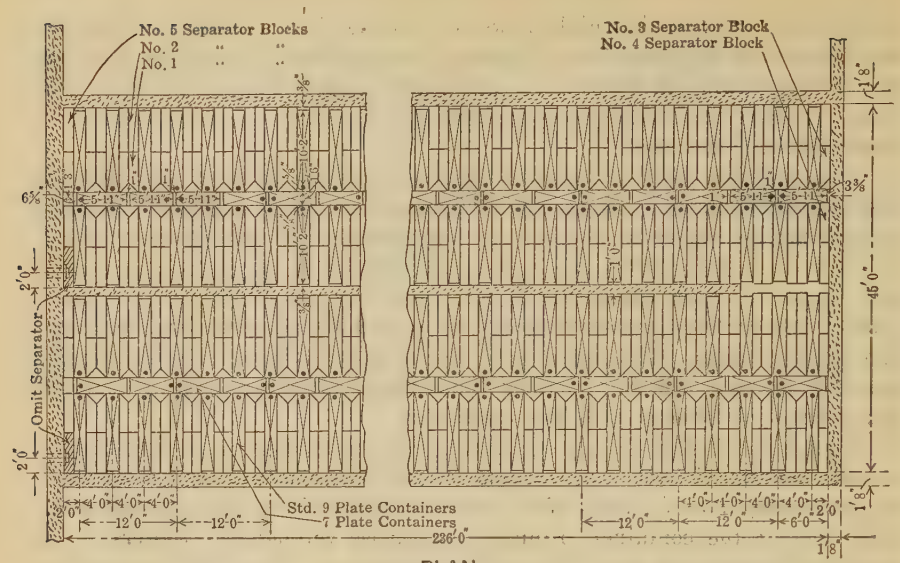
The supply of air to each unit enters at the end adjacent to the side operating galleries, through a 12-in. pipe, and is measured by an air meter. The 12-in. pipes pass through the gallery walls, run downward, and branch into two 10-in. pipes directly over the center of the 12-in. partition walls in the center of the aeration units. The 10-in. air-lines are carried the entire length of each compartment and are supported on top of the precast container separator blocks. Each diffuser-plate container is supplied with air through a 1½-in. pipe connecting with the 10-in. air-header.

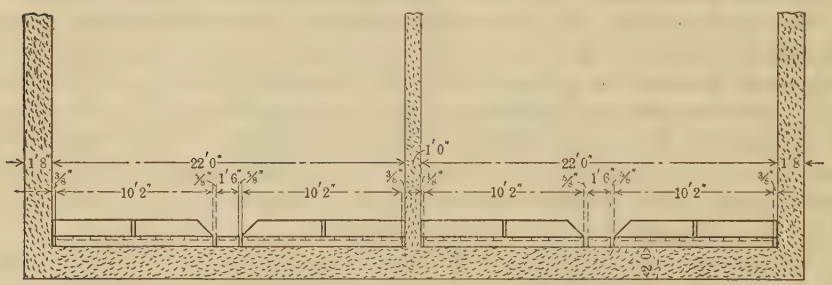
All the information pertinent to the performance of the tanks is plainly set forth on the recording mechanism, and the control valves are within easy reach of the operator.

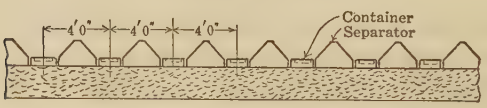
Settling Tanks

As stated previously, an aeration unit has great flexibility, and high rates of flow can be maintained without impairing the character of the effluent. A settling tank likewise has flexibility, but of a different character and to a lesser comparative degree.

The limit of the capacity of a settling tank for a given sludge is reached, when the sludge in the tank is carried over the sides of the effluent troughs and out with the effluent. Experience in operating settling tanks at the testing station led to the adoption of 1 600 gal. per sq. ft. per day as the safe rate on which to base the number and size of tanks. This basic figure represents the number of gallons of sewage which can be settled every 24 hours on each square foot of horizontal liquid surface in the tanks, and does not include the return sludge in the mixture which enters the tanks. Therefore,







236	9 Plate Containers	per Tank	5664	for Total of 24 Tanks
78	5 " "	" "	1872	" " " " "
232	No. 1	Separators " "	5568	" " " " "
232	No. 2	" " "	5568	" " " " "
10	No. 5	" " "	240	" " " " "
2	No. 4	" " "	48	" " " " "
2	No. 3	" " "	48	" " " " "

SEWAGE DISPOSAL PLANT
SEWERAGE COMMISSION OF THE
CITY OF MILWAUKEE
PLAN SHOWING
CONTAINER & SEPARATOR SETTING
IN AERATION TANK

FIG. 1.

when carrying 20% of sludge in the plant, the total volume of liquor entering the settling tank per square foot of surface area is 2 000 gal. Of this volume 1 600 gal. represent the sewage flow and the remaining 400 gal., the 20% of activated sludge.

Table 1 shows that the average and maximum expected flows for 1930 are 85 183 000 gal. per day and 165 610 000 gal. per day, respectively, the maximum

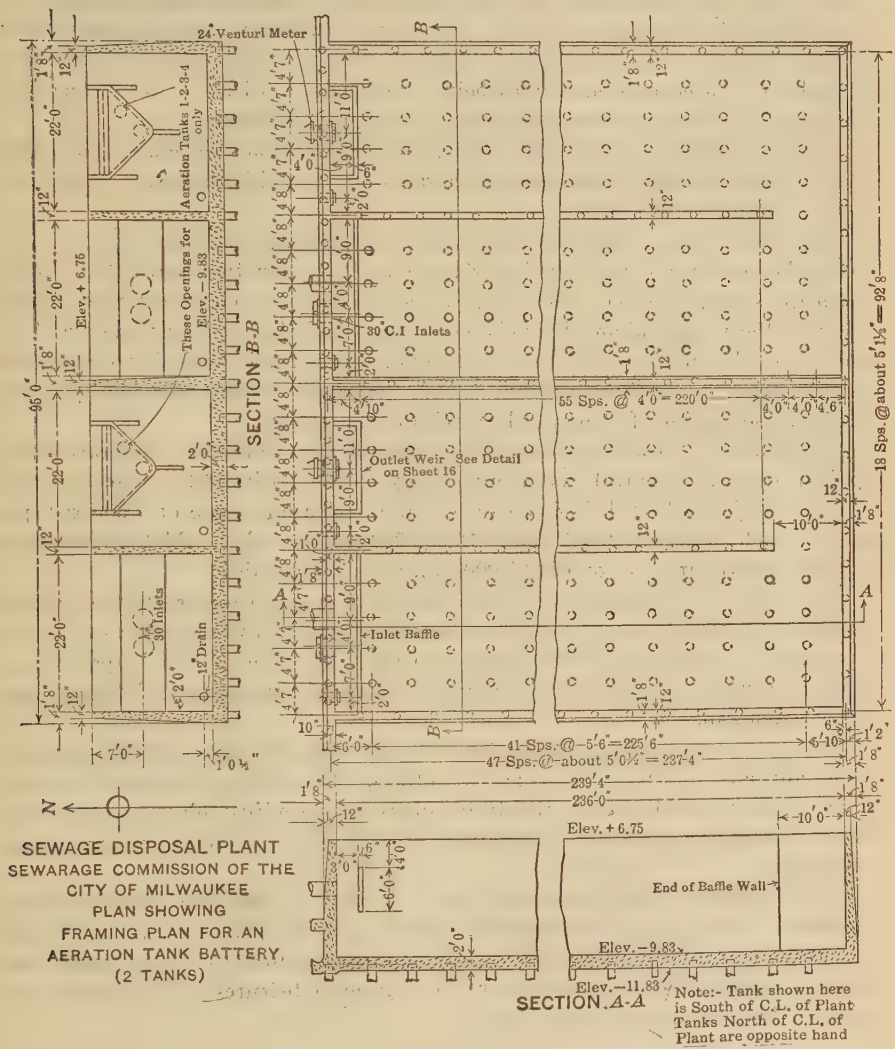

24-Venturi Meter
Aeration Tanks 1-2-3-4 only
These Openings for Elev. — 9.83
Elev. + 6.75
30 C.I Inlets
SECTION B-B
55 Sps. @ 4'0" = 220'0"
Outlet Weir See Detail on Sheet 16
18 Sps. @ about 5'1½" = 92'8"
Inlet Baffle
30 Inlets
12" Drain
41-Sps.-@-5'6"=225'6"
47-Sps.-@-about 5'0½" = 237'4"
239'4"
236'0"
Elev. + 6.75
End of Baffle Wall
Elev. — 9.83
Elev. — 11.83
SECTION A-A
Note:- Tank shown here is South of C.L. of Plant Tanks North of C.L. of Plant are opposite hand
SEWAGE DISPOSAL PLANT
SEWARAGE COMMISSION OF THE CITY OF MILWAUKEE
PLAN SHOWING
FRAMING PLAN FOR AN AERATION TANK BATTERY,
(2 TANKS)

FIG. 2.

flow being about 94% greater than the average flow. This great difference between the average and maximum flow makes it necessary to provide settling tanks for the maximum flow rather than for the average, because if the settling tanks were proportioned to the average flow on a basis of 1 600 gal. per sq. ft., during the maximum flow this quantity would become 3 100 gal.

per sq. ft., which is greatly in excess of the safe rate. Rates as high as 2 000 gal. per sq. ft. have been obtained in the tanks at the testing station and a similar rate in the large tanks now being designed can reasonably be expected. However, a study of the settling-tank data, indicates that the rate decided on is conservative and on the side of safety.

A depth of 15 ft. from the liquid surface to the inside bottom of the tank was adopted and considered the depth necessary to allow adequate storage for the sludge during maximum flow, based on a constant rate of sludge withdrawal. Each of the eleven tanks (Plates VI and VII) is octagonal in shape at the top, the corners and sides being sloped down with a 2 to 1 slope and converged to form the circular bottom, 98 ft. in diameter.

The mixed liquor enters the tanks from opposite sides through eight gates, 8 ft. by 9 in. in size. There are four gates on each side, arranged to operate together or individually.

On the inside of the tank, the center of the gate openings is 4 ft. 6 in. from the top of the tank walls, or allowing 1 ft. for free-board, 3 ft. 6 in. from the sewage surface. The part of the wall above each gate forms a baffle between the mixed liquor channels which surround the settling tanks on three sides, and the inside of the tanks. The function of this baffle is to cause the mixed liquor to enter the tanks below the upper clear-water zone and to shorten the length of downward travel for the sludge.

Each tank is to be equipped with complete Dorr thickener apparatus controlled from the central operating gallery. The horizontal surface area of the sewage in each tank is 8 550 sq. ft., and, based on a rate of 1 600 gal. per sq. ft., each tank will treat about 13 680 000 gal. per day, which corresponds to an approximate rate of 68 000 000 gal. per acre per day.

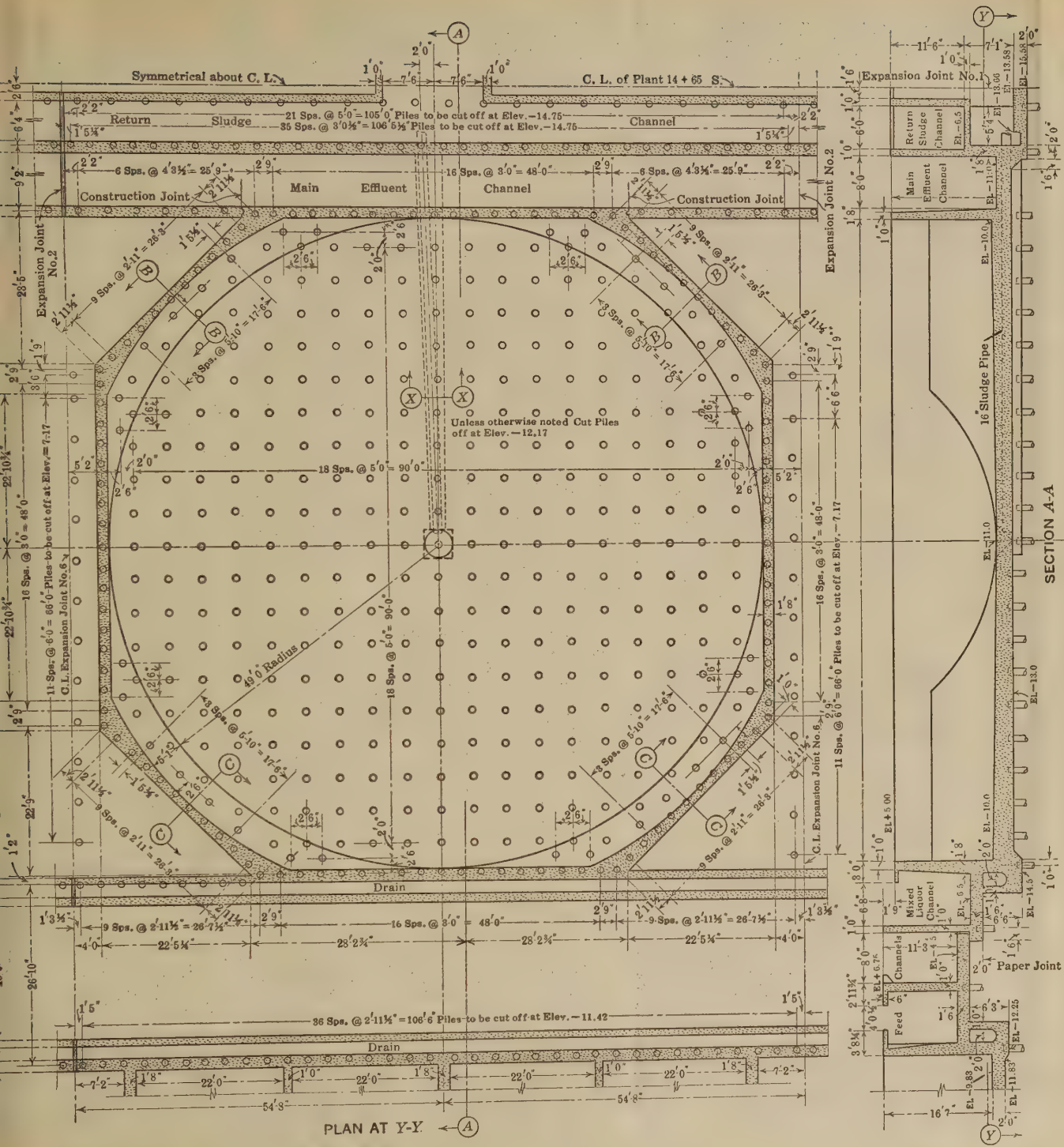
The effluent is drawn off at the surface through three rectangular steel troughs extending across the tanks at right angles to the direction of flow in the tanks. These troughs discharge into the main effluent and by-pass channels running east and west through the center of the plant, and thence into Lake Michigan.

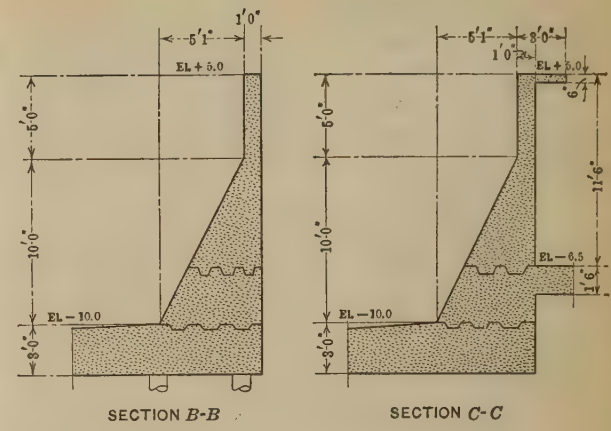
Sludge is drawn from the bottom of each tank through a 16-in. cast-iron pipe laid horizontally in the concrete bottom. The sludge is carried by 14-in. riser pipes, vertically, from the 16-in. pipes to the adjustable pipe-regulating arms, through which it is discharged directly into the main return sludge channels.

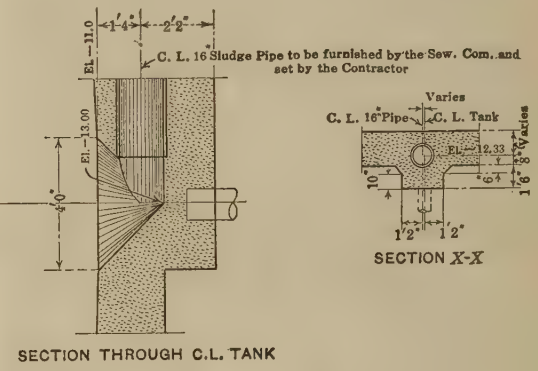
The settling tanks marked 1, 2, 3, 4 and 5, and the aeration units marked 1, 2, 3, and 4, are designed and arranged so that they can be isolated to serve as units for the special treatment of sludge to be pressed, should such special treatment become necessary. The general features of these tanks are similar to those already described.

Channels

The ratio of diffuser-plate area to the area of the sewage surface in the feed and mixed liquor channels is approximately the same as that in the aeration tanks, about 1 to 4, the return sludge channel being provided with a ratio of about 1 to 6.


Symmetrical about C. L.
C. L. of Plant 14 + 65 S.
Return Sludge Channel
21 Sps. @ 5'0" = 105'0" Piles to be cut off at Elev. -14.75
35 Sps. @ 3'0½" = 106'5½" Piles to be cut off at Elev. -14.75
6 Sps. @ 4'3½" = 25'9"
16 Sps. @ 3'0" = 48'0"
6 Sps. @ 4'3½" = 25'9"
Main Effluent Channel
Construction Joint
Construction Joint
Expansion Joint No.2
Expansion Joint No.2
Expansion Joint No.1
Return Sludge Channel
Main Effluent Channel
16" Sludge Pipe
Unless otherwise noted Cut Piles off at Elev. -12.17
18 Sps. @ 5'0" = 90'0"
49'0 Radius
C.L. Expansion Joint No.6
11 Sps. @ 6'0" = 66'0" Piles to be cut off at Elev. -7.17
16 Sps. @ 3'0" = 48'0"
C.L. Expansion Joint No.6
Drain
9 Sps. @ 2'11½" = 26'7½"
16 Sps. @ 3'0" = 48'0"
9 Sps. @ 2'11½" = 26'7½"
36 Sps. @ 2'11½" = 106'6" Piles to be cut off at Elev. -11.42
Drain
54'8"
54'8"
Paper Joint
Mixed Liquor Channel
Channels
Feed
PLAN AT Y-Y.
SECTION A-A




C. L. 16" Sludge Pipe to be furnished by the Sew. Com. and set by the Contractor
SECTION THROUGH C.L. TANK
Varies
C. L. 16" Pipe
C. L. Tank
EL - 12.33
SECTION X-X

NOTE:- Tank shown here is South of C. L. of Plant
Tanks North of C. L. are opposite hand

N

SEWAGE DISPOSAL PLANT

SEWERAGE COMMISSION OF THE
CITY OF MILWAUKEE.
PLAN SHOWING
SEDIMENTATION TANK FRAMING
TANKS 6 TO 13
PLAN AND SECTIONS

The effective depth of all channels is 10 ft. This depth permits the drainage system of the plant to be constructed under the channels, practically no additional excavation being required.

The value of the aerated channels, as aeration units, was considered at length, but owing to the variables, such as quantities, velocities, length of travel, time of detention, etc., which entered into the problem, it was decided finally to credit to the aeration plant as a safety factor whatever benefits were derived from aeration in the channels.

The feed channels—two on each side of the plant at the head of the rows of aeration tanks—are provided in duplicate in order to insure against shutting off parts of the plant should it become necessary to drain a channel for the purpose of making repairs. All feed channels are designed for the anticipated 1950 flow.

The mixed liquor channels are arranged so that sections can be cut out of service and drained for the purpose of making repairs, etc. Each section is comparatively small and with only one section between two settling tanks out of service, it is impossible to cut out of service more than one entire tank. Owing to the probable infrequency of having to make repairs in the mixed liquor channels, stop-planks are to be used instead of sluice-gates. All the feed channels and mixed liquor channels are 8 ft. wide.

The return sludge channels, each 6 ft. wide and 11 ft. deep, are in duplicate for reasons already mentioned in connection with feed channels. They receive the discharge of sludge from the settling tanks and will contain a maximum depth of 5.5 ft. of sludge, which will permit visual inspection of the sludge being discharged from each settling tank. At the extreme west end of the plant, the two channels, each of which are equipped with sluice-gates for control purposes, unite into a small forebay at the head of a 54-in. pressure conduit which carries the sludge to the return sludge pumps.

The two main effluent channels, each 8 ft. wide and 16 ft. deep, for obvious reasons, are not equipped with diffuser plates. The depth of water in these channels will depend on the stage of the water in Lake Michigan. At Elevation 0.00, the depth will be about 11 ft. The parts of the channels between the end of the plant and the outer bulkhead are designed as pressure conduits in the form of inverted siphons and at a sufficiently low elevation to permit of future plant extensions directly on top. The discharge from the plant will occur at the outer face of the bulkhead in about 12 ft. of water.

Plant Hydraulics

During the early stages of the design of the disposal plant, the engineers were confronted with the fact that to convey large and fluctuating quantities of sewage in and around the proposed plant and to and from the various units, and, at the same time, provide self-cleansing velocities for a mixture of sewage and activated sludge, would require a head greatly in excess of that available.

It became necessary, therefore, to devise methods whereby satisfactory flow conditions could be obtained and the hydraulic losses confined within the fixed limits. Equipping all channels conveying sewage mixed with activated sludge

with diffuser plates, and supplying sufficient air (approximately 1 cu. ft. of air per sq. ft. of diffuser plate per min.) for proper agitation and the prevention of deposition of solids, appeared to be the most practical solution, and was adopted. Besides affecting a substantial reduction in loss of head in the main feed channels, this method possessed the additional advantage of insuring an intimate mixture of sewage and sludge prior to its entering the aeration units.

Following the determination of the major items connected with the design of the plant, such as the most desirable arrangement of units, channels, etc., also flow intensities, size of units, and operating facilities, the problem of analyzing flow conditions and computing and distributing hydraulic frictional losses was considered, as follows: To proportion all channels, conduits, pipes, gate-openings, weirs, and sewage-measuring apparatus in the most advantageous manner, confining the total frictional loss through the plant to 4.25 ft., based on maximum assumed conditions of flow hereinafter fully described. This total hydraulic loss represents the difference between the hydraulic elevation of the sewage at the head of the fine screen tail channel, which is +7.25, and the elevation of the assumed high water in Lake Michigan, which is +3.00, based on city datum.

The determination of frictional losses through the fine screens, grit-chambers, coarse screen house, and connecting channels, will not be discussed, as it is outside the scope of this paper.

Elevation + 3.00 which was used as the maximum stage of the water in Lake Michigan, was determined from an thorough study of Lake level records taken over a period of fifty years, and although it is questionable whether this maximum stage will repeat and continue over an extended period, it is believed that the use of this elevation will insure the uninterrupted operation of the plant against all except the most extraordinary and unusual occurrences. For several years past, the Lake level has fluctuated between Elevations 0.00 and — 1.00.

Mixing Channel.—The plant described in this paper is assumed to start at the head of the fine screen tail channel, which is the point of application of the activated sludge to the sewage flow and also the point of beginning for channels equipped with diffuser plates. All channels leading up to this point, and also the main by-pass channel which is directly beneath the main sewage channel and extends throughout the entire length of the plant, are not equipped with diffuser plates for obvious reasons.

The part of the channel between the fine screens and the gate-house at the plant entrance (the channel between *A* and *B*, Plate VI), and which is termed the mixing channel, is 9 ft. deep and 14 ft. wide and is equipped with diffuser plates in the ratio of 1 to 4, as are all the other channels except those conveying sludge from the settling tanks.

The initial mixing of sewage and returned sludge occurs in this channel prior to its diversion into and through the four feed channels which supply the aeration units. This channel is proportioned to accommodate a maximum flow at the rate of 231 010 000 gal. per day, which is the estimated ultimate capacity of the 1950 plant, plus 40% of returned sludge, based on the esti-

mated 1950 average flow of 128 775 000 gal. per day, making a total of 316 860 000 gal. per day. The 40% of returned sludge referred to, will be explained later under the heading, "Return Sludge Channels."

The following values of Kutter's n were used for the determination of frictional losses:

$n = 0.013$ for pipes.
$n = 0.020$ for aerated channels.
$n = 0.015$ for unaerated channels.

Head losses through gates, openings in walls, and at the entrance to pipes, in most instances, have been based on velocity and entrance-head determinations, the entrance head being taken usually as one-half the velocity head, due credit being given wherever feasible to the velocity of approach.

Considerable uncertainty existed regarding the selection of a proper friction coefficient for aerated channels, owing to the character of the bottoms which, when equipped with diffuser containers and container separators, will offer considerable resistance to the flow, and also owing to the upward passage of diffused air through the mass of sewage, with the attendant possibility of setting up counter currents.

The consensus of opinion, however, was that the coefficient adopted ($n = 0.020$) at least would give results on the side of safety.

Gate-House.—The gate-house, so-called because in it are four, 6 by 10-ft. control sluice-gates, one at the entrance to each of the four sewage feed channels, is located at what is called herein the entrance to the plant and also at the terminus of the mixing channel. Mixing is continued, however, throughout the entire length of each feed channel.

For reasons explained previously the feed channels are in duplicate. During the time the channel is out of service, the aeration tanks will take their supply from the duplicate. For the purpose of assuming a maximum condition on which to determine the loss of head through the head-gates, it was considered possible that the maximum flow of sewage might occur at a time when sludge was being returned to the plant at double the normal rate, at which time two of the feed channels, one on each side of the plant, were out of service, leaving the two remaining active channels to pass the entire flow.

This assumed condition may appear to be severe, but as the total loss involved was not considered to be excessive, no material advantage would have resulted from the adoption of less severe conditions.

Feed Channels.—Each of the four sewage feed channels is 8 ft. wide and 10 ft. deep (depth of sewage), and under normal plant operation, all of them will be in use at the same time.

Each of the twenty-four aeration units is equipped with duplicate inlet pipes, 30 in. in diameter, and sluice-gates, each inlet pipe taking its supply from one of the duplicate feed channels.

The total length of each pair of channels is about 790 ft. for the 1930 plant and 1 120 ft. for the 1950 plant, it being assumed that the extension required to accommodate the estimated sewage flow for 1950 will consist of

twelve additional aeration units and six sedimentation tanks, or an increase of about 50% of the capacity of the plant designed for the 1930 period.

The maximum conditions of flow on which loss of head determinations was based, are those described previously, in connection with the functions of the gates in the gate-house. The frictional loss was computed on an increment basis, that is, the total quantity carried in the channel was decreased at uniform intervals throughout its length by the quantity supplied to each aeration unit, and a separate computation was made for the decreased flow in each succeeding increment.

The reason for considering the channels immediately adjacent to each row of aeration tanks in determinations for the loss of head, was that the cross-section of the channel is restricted by the 30-in. inlet pipes which connect the duplicate feed channel to the aeration units, and the loss of head would be slightly greater in this channel than in the duplicate, on account of this restriction.

Aeration Units Nos. 1 and 2 were assumed to be out of service for the purpose of simulating a possible condition in operation whereby the flow, which under ordinary operation would enter and pass through Units Nos. 1 and 2, would be required to pass to Tanks Nos. 3 and 4 and thereby cause a small head loss which would not occur with Units Nos. 1 and 2 in service.

Aeration Units.—Each of the twenty-four aeration units is designed on the basis of treating sewage at an average rate of 15 000 000 gal. per acre per 24 hours for the estimated 1930 flow.

Due to the fact that this rate is conservative and that it is possible to operate safely at rates to and including 20 000 000 gal. per acre per day, it is to be assumed that the aeration plant would be operated to its limit before new units were added to accommodate future conditions of increased flow. This assumption indicates the necessity for basing hydraulic loss determinations on the maximum flow conditions existing when operating at a rate of 20 000 000 gal. per acre per day, instead of on similar conditions for a rate of 15 000 000 gal. per acre per day.

When operating on an average rate of 20 000 000 gal. per acre per day, an aeration unit will treat approximately 4 774 000 gal. per day; the total quantity of the liquor entering the tank daily through the two 30-in. inlet pipes, including 20% of return sludge, is approximately 5 967 500 gal.

The maximum daily rate of flow for the 1930 period, as shown in Table 1, is about 94% greater than the average flow, which means that the peak flow would be at the rate of 9 261 600 gal. per day. The total quantity of liquor passing through the tank at this rate, including return sludge at the normal rate, would be about 10 455 100 gal. It is on this latter quantity that the loss of head through the 30-in. inlet pipes which connect the feed channels and aeration units, was determined; and for the purpose of establishing a maximum condition, this total flow was assumed to pass through one 30-in. inlet only, that connected with the feed channel farthest removed from the aeration units. The head loss incurred in the passage of the liquor through the aeration units is negligible.

The outlet end of each aeration unit, as described previously, is equipped with an overflow weir, and possesses suitable adjusting facilities for obtaining the proper setting. The flow passing over this weir discharges into a forebay which is connected to the mixed liquor channel by a 24-in. Venturi meter, equipped with a sluice-gate, and all the necessary apparatus for measuring and recording the flow leaving the unit. With the facilities thus provided for measuring the flow, each aeration unit should be susceptible to close regulation and reasonably complete control.

The fact is recognized that although each and every aeration unit is designed and will be constructed in precisely the same manner and, theoretically, should treat like quantities under given conditions, it is quite probable that each unit will have its own peculiar characteristics, and to secure the most efficient results, will have to be operated accordingly. The entire field of meters and measuring devices for measuring liquids was thoroughly investigated before the Venturi meter was finally adopted.

Mixed Liquor Channels.—The hydraulic losses occurring in the mixed liquor channels are derived from the assumption that a quantity of sewage equivalent to the rated capacity of one sedimentation tank will be required to pass through two of the 4 by 6-ft. openings in the partitions which are placed at regular intervals in the mixed liquor channels for the purpose of isolating sections of the channel by the use of stop-planks.

The loss of head caused by the actual flow of sewage in the channels which are 8 ft. wide and 10 ft. deep (depth of liquor), is practically negligible because of the frequency of the points of discharge from the aeration units, resulting in short lengths of travel and small quantities, as compared with the conditions of flow and quantities carried by the feed channels.

Based on conditions creating loss of head alone, it would have been possible to use a narrower channel for the mixed liquor; but the relative importance of maintaining the elevation of the surface of the liquor in all parts of the channels as nearly constant as possible for ordinary conditions of operation, was deemed of greater moment than the small saving in money which could have been effected through the use of smaller channels.

Sedimentation Tanks.—The rated capacity of each of the 98-ft. sedimentation tanks, as mentioned previously, is 13 680 000 gal. per day. This quantity, together with the return sludge content, will enter each tank through the eight 8 ft. by 9-in. sluice-gates, four each on opposite sides of the tank. With the gates fully open, the frictional losses when passing the quantity mentioned will be negligible.

There is a remote possibility that some manipulation of gates will be required to take up slight differences in the elevations of the edges of the effluent trough weirs of the sedimentation tank, and the surface of the sewage in the mixed liquor channels. A small allowance in head, therefore, has been made for a possible condition requiring throttling of the gates.

Effluent Troughs.—The top edge of the weir plates will be set at the proper elevation and leveled when the tanks are first placed in service.

Theoretically, assuming that the flow over each of the six weirs in a tank would be equal to one-sixth of the total flow through the tank, the

corresponding depth of flow over each weir would be equivalent to that produced by 13 680 000 gal. passing over a weir 588 ft. long, or about $\frac{5}{8}$ in. It is questionable whether the theoretical conditions cited would obtain in the case of the three troughs and, in view of the lack of information pertinent to the possible skimming action of multiple troughs, it was thought best to make a somewhat greater allowance in the quantity passing over both weirs of the middle trough. Therefore, an allowance of 50% was made, in lieu of the 33⅓% used in the theoretical trial calculation.

Effluent Channels.—The two effluent channels which are parallel and adjacent to the return sludge channels, on opposite sides of the center line of the plant, receive the effluent directly from the ends of the effluent troughs and convey it to an outlet chamber at the eastern bulkhead, as indicated on Plate VI, where it is discharged directly into Lake Michigan through submerged openings which are in duplicate.

The channels are to be constructed as open channels as far as the eastern end of the 1930 plant and, from this point to the Lake, as pressure conduits.

The function of the pressure conduits, besides that of conveying the effluent to Lake Michigan, is twofold: First, to minimize in the open channels the effects due to "swell" and wave action caused by on-shore winds; and, second, to permit the continuance of the construction of future plant in an easterly direction.

It is intended that the effluent from the future extensions to the plant shall flow west instead of east, as in the 1930 plant, and discharge into the submerged chambers provided at the entrance to the pressure conduits, the latter being proportioned to accommodate the maximum estimated 1950 flow.

Each of the effluent channels is 8 ft. wide and 16 ft. deep (total depth), and each of the effluent pressure conduits is 8 ft. wide and 5 ft. deep.

By-Pass Conduit.—The by-pass conduit which begins at the terminus of the riser shaft of the intercepting sewer in the coarse screen house, is to be constructed directly beneath the main sewage conduit and will extend throughout the length of the plant, as mentioned previously, affording by-pass facilities for the coarse screen house, grit-chamber, and fine screens. It will also connect with the two effluent channels at a point beneath the mixing channel at the gate-house.

The maximum loss of head in the effluent channels and pressure conduits will occur when all the sewage is by-passed around all the aeration units and sedimentation tanks (a condition which it is hoped will never exist), but as the loss of head due to this condition would occur at the time when the plant would be out of service, the head losses here considered and adopted are those resulting from the determinations based on the maximum discharge of effluent from the sedimentation tanks.

Sedimentation Tanks Nos. 1, 2, 3, 4, and 5, at the western end of the plant, were assumed to be out of service in making the determination for loss of head.

Return Sludge Channels.—The head loss in the two return sludge channels is entirely separate and apart from the losses which effect the flow through

the plant, and was given separate consideration from the standpoint of conserving the head and thus reducing the lift for the return sludge pumps.

Each sedimentation tank is equipped with two adjustable discharge pipes, 14 in. in diameter. The double discharge feature affords facilities for discharging the sludge into either of the channels.

It is not planned to operate both discharge pipes at the same time, but the design does contemplate the use of both channels at all times, except when it might become necessary to replace a broken filtros plate, repair a pipe, or perform other work of a similar nature in one or the other of the channels.

The assumed condition producing the maximum loss of head will occur when one of the channels is out of service and sludge is being withdrawn from each tank at double the normal rate, and the total quantity is being carried in one channel. The calculation for loss of head was based on this assumed severe condition in order to determine the lowest possible elevation to be used as a basis in fixing the extreme lower limit for the lift for the return sludge pumps. The total length of channel considered in the determinations for loss of head is equivalent to that required to extend to the extreme eastern limits of the 1950 plant.

The aerated return sludge channels terminate at the gate-house, and the sludge carried by them is discharged at this point into an unaerated forebay located at the entrance to an unaerated pressure conduit. This conduit (54 in. in diameter) is proportioned so that the expected minimum flow of sludge through it will create a velocity somewhat in excess of 2 ft. per sec. for the 1930 quantity. The flow conditions on which the head losses are based are the same as those described for the channels.

The return sludge pumping equipment will consist of three motor-operated centrifugal pumps, each capable of pumping sludge at rates of 6 000 000, 9 000 000, and 12 000 000 gal. per day. It will draw its supply from the suction well mentioned previously and force it through a 48-in. cast-iron pipe equipped with a Venturi meter to the head of the fine screen tail channel (mixing channel), where it will be discharged through multiple outlets into the sewage entering the plant.

The sludge to be pressed, or waste sludge as it is called, will be taken from the 48-in. force main, or directly from the aeration or sedimentation tanks which may be reserved for the special treatment of waste sludge, and conveyed through a conduit to the sludge de-watering plant.

Sludge Draw-Off Pipes.—Sludge is to be drawn from the sedimentation tanks by means of hydrostatic head and, unless the actual operation of the plant for a period of time shows indications to the contrary, it will be withdrawn at practically a constant rate throughout each 24 hours.

Although the designs are based on a constant sludge return of 20% and facilities are provided at the same time in the sludge draw-off pipes and in the sludge pumping equipment for a variable rate of sludge return, it should not be inferred that a variable rate of return is contemplated.

The basic figure of 20% and the quantity of sludge corresponding thereto are used in the sense that they represent a probable high average rate, possibly close to the rate which, in actual operation of the plant, may prove to be

the maximum required. Seasonal changes, as applied to the character of the sewage, may indicate the necessity for varying somewhat the rate and percentage of sludge return and, in the light of past experience, in connection with the operation of the testing station plant, the indications appear to be that the basic figure of 20%, if varied at all, will be reduced rather than increased.

The facts leading up and entering into the determination of necessary provisions relative to adequate and proper flexibility for devices and apparatus used in connection with the manipulation of return sludge, can be summed up, as follows:

First.—Approximately seven years will elapse between the time of placing the plant in operation and the 1930 period for which it is designed.

Second.—How closely the actual sewage flow in 1930 will correspond to that which has been estimated, is problematical.

Third.—The actual rate or rates of returning sludge can be determined definitely for the sewage flow from the entire city only after the plant has been placed in operation.

Fourth.—There may be occasions, possibly periods of short duration, when sludge will have to be drawn from the tanks at excessive rates. This may be required in order to lower the sludge level in one or more, or possibly all, of the tanks. The likelihood of this condition arising would appear to depend largely on the efficiency of the operation resulting from the personal equation.

The draw-off pipes are 16 in. in diameter on the horizontal, and 14 in. on the vertical run, except those for the 43-ft. tanks, which are 8 in. in diameter, horizontally and vertically. There is a cone-shaped entrance to the 16-in. pipes in the bottom of each tank, at the center, where the sludge which has been deposited on the bottom and which has been brought to the center by the spiral ploughing action of the Dorr thickeners, enters the draw-off pipes and is carried to the return sludge channels by hydrostatic pressure.

The quantity of sludge being withdrawn is controlled by the difference in head between the elevation of the sewage surface in the tanks and that of the ends of the adjustable 14-in. discharge pipes which are to be operated from floor stands through the medium of worms and circular racks.

In selecting the proper size for the draw-off pipes, the determining factors were the minimum velocity and the maximum loss of head, the minimum velocity being established at 2 ft. per sec. and the maximum loss of head at 3½ ft. An allowance of 1 ft. was made between the centers of the discharge pipes and the maximum elevation of the sludge surface in the return-sludge channels, the total distance from the liquid level in the sedimentation tanks to the maximum liquid level in the sludge channels being established as 4½ ft. This allowance will permit unobstructed visual inspection of the flow of sludge from the discharge pipes, and the depth of sludge in the sludge channels, namely, 5½ ft., referred to previously, is sufficient to submerge properly the entrance to the sludge conduit in the forebay, at the western end of the channels.

If all the sedimentation tanks are placed in service during the early years of the plant's operation, the indications are that velocities slightly less than 2

ft. per sec. will obtain in the 16-in. pipes. If, on the other hand, as a matter of operation, the number of tanks in use for a given flow is proportional to the flow, based on the practical limit of the tank's capacity, it would appear reasonable to assume that the pipe velocities would be susceptible to the same range of control which would obtain when all the tanks are in service and operating on the flow for which the plant as a whole is designed.

The 16-in. pipes are to be cast in the concrete forming the floors of the sedimentation tank and are sufficiently large to accommodate the maximum assumed rates of sludge flow based on sedimentation rates considerably in excess of 1 600 gal. per sq. ft., without causing the frictional losses to exceed the fixed limit. The 14-in. vertical pipes and adjustable discharge pipes which can be erected independent of the tanks and walls, in combination with the horizontal 16-in. pipes, will accommodate rates of sludge flow corresponding to sedimentation rates to and including 1 600 gal. per sq. ft., without exceeding the allowable friction loss. Should future requirements so demand, the 14-in. pipes can be removed and a larger size substituted.

The waste sludge has been referred to only casually, for the reason that it is in sufficient quantity to influence the conditions of flow through the plant. For example, if the daily accumulation of sludge in the plant is at the rate of 15 000 gal. of 98.5% moisture material per 1 000 000 gal. of sewage treated, the total daily quantity for the 1930 period will be about 1 275 000 gal. and for the 1950 period about 1 920 000 gal. It can be readily seen that this comparatively small quantity, when distributed over the entire plant, would have no appreciable effect on any calculations pertaining to the plant hydraulics.

Air-Distribution System

The compressor plant for furnishing air will consist of four, 30 000 cu. ft. per min., Ingersoll-Rand turbo blowers direct connected to Allis Chalmers steam turbines. There will be three active units and one spare.

No attempt will be made by the writer to give a detailed description of the proposed compressor and power plant. Only a few references will be made to the arrangement, character, and capacity of the machinery. Each blower will take its supply of air from the outside atmosphere through screened louvers along the east side of the power house and just below the roof.

The air will pass downward to the blowers through 36-in., cast-iron, inlet pipes equipped with Venturi meters and spray washers for measuring and washing the volume of free air to be compressed. After being compressed, the air will be discharged through 30-in., cast-iron, outlet pipes and enter the main air-header leading to the plant. The capacity of each unit should be interpreted as meaning 30 000 cu. ft. of free air per min. compressed to 10 lb. per sq. in.

The air requirements for the 1930 plant are based on supplying air to the aeration tanks at the rate of 1½ cu. ft. of air per gal. of sewage treated. The conversion of this figure to a single diffuser-plate basis, assuming that sewage is being treated at the rate of 15 000 000 gal. per acre per day, with a diffuser ratio of 1 to 4, results in a figure which indicates that each diffuser plate will pass air at the rate of 1.48 cu. ft. per min.

Each aeration unit will actually contain 2 514 diffuser plates, and if each plate is supplied with air at the rate mentioned, the total air required per minute per aeration unit will be about 3 820 cu. ft. and, for the twenty-four units, about 90 680 cu. ft.

Air will be supplied for agitation to the diffuser plates in all the aerated channels, at the approximate rate of 1 cu. ft. of air per diffuser plate per min. The total number of plates in the channels is 11 520 and will require about 11 520 cu. ft. of air per min. The total quantity of air required for the aeration tanks and channels will be at the approximate rate of 103 200 cu. ft. per min. It will be noted that this quantity exceeds the combined rated capacity of the three active blowers which is 90 000 cu. ft. per min. at a pressure of 10 lb.

The secondary rating of the blowers, which is approximately 35 000 cu. ft. of air per min. at a pressure of 8 lb., indicates an available combined capacity of 105 000 cu. ft. per min. for the three units. This capacity is more than ample to satisfy the air requirements for the plant, and it is questionable whether the pressure losses in the air-distribution system as a whole will ever exceed 8 lb. per sq. in.

Experience in operating the plant at the testing station has shown conclusively that when foreign substances, such as dirt and oil, are eliminated from the air supply, there is little danger to be anticipated from the clogging of diffuser plates and the resulting increase in pressure.

The equipment which has been selected and purchased for compressing and washing the air, was designed and constructed under specifications which required a 100% performance relative to the quality of air delivered. The importance of a constant clean air supply is vital to the operation and maintenance of an activated sludge plant and should not be under-estimated.

The problem of designing an adequate and practical piping system for the distribution of air was given a great deal of study and consideration and was finally developed from the following basic conclusions.

First.—That the drop in pressure due to frictional losses in the pipes should not exceed ½ lb., based on the estimated requirements for the 1950 plant. This allowance does not include the fixed loss through the check-valve and gate-valve on each blower outlet pipe and the loss through the diffuser plates.

Second.—That accurate and convenient facilities should be provided for measuring the volume of air supplied to each aeration unit and to the mixed liquor, feed, and return-sludge channels.

Third.—That the pressure drop in the aeration units should be based on the quantity of air required to treat sewage at the rate of 20 000 000 gal. per acre per day, allowing 1½ cu. ft. of air per gal. of sewage, or 1.98 cu. ft. of air per diffuser plate per min.

General Arrangement of Air-Piping.—One of the first steps in the design was to give separate consideration to the depths of liquor in the aeration units and in the channels. The depth of liquor in the aeration units, as noted previously, is 15 ft.; that in the feed and mixed liquor channels is 10 ft.; and that in the return-sludge channels is about 5½ ft. maximum.

The largest proportion of the total quantity of air is used in the aeration units. It is here, also, that the maximum pressure is required, owing to the depth of the liquor.

It was decided, therefore, after having compared the cost of providing and operating separate low-pressure blowers for supplying air to the channels with the cost of supplying air at an excessive pressure from the large units, that better economy could be obtained through the use of a system of sub-headers carrying pressure suitable to the depth of liquor, the supply of air being taken from the main air-headers and the reduction in pressure obtained through resistance in pressure-reducing valves.

The two sub-headers which supply the feed and mixed liquor channels with air, are connected to the two main air-headers at points about opposite the center of the middle units of the 1950 plant, the connection pipes, which are 12 in. in diameter, being equipped with pressure-reducing valves and air meters.

The sub-header supplying air to the return sludge channels and the mixing channel is connected to the main air-header at the western end of the plant, in the gate-house. This connection which is 8 in. in diameter, is also equipped with a pressure-reducing valve and an air meter.

Each aeration unit will take its supply of air directly from the main air-header, the connection being made at the top of the header in order to avoid carrying condensate into the piping of the aeration unit.

The connecting pipes between the main air-headers and the piping in the aeration units are 12 in. in diameter, and each pipe is equipped with a gate-valve and an air meter.

Determining the Air-Pipe Sizes.—Research work carried on in connection with securing data and information pertinent to the allowance of adequate pipe capacity for the air, led to the adoption of the "Thorkelson" formula. This formula may be expressed as follows:

$$P = \frac{L\,V^2}{25\,000\,B}$$

in which,

P = loss of pressure, in ounces per square inch.
L = length of pipe, in feet.
V = velocity of air, in feet per second.
B = diameter of pipe, in inches.
25 000 = constant.

Twelve diameters of pipe are allowed for bends of 90 degrees.

The air compression in the blowers will occur under adiabatic conditions, and the compression factors used for determining compressed volume, velocity, and the resulting pressure drop, were based on this condition.

It is quite likely that some of the heat of compression contained in the air will be lost in the air mains through radiation, resulting in loss of volume, less velocity, and less friction. Possibly, however, this loss will be made up by the heat resulting from the friction of the air against the walls of the pipe, and, consequently, the compression factors used were not corrected for heat losses resulting from radiation.

The allowable pressure drop of 8 oz., which was used as a basis in proportioning the pipe sizes, was determined from cost comparisons (Table 2) prepared to indicate the probable relation between the cost of compressing air and that of enlarging the pipe sizes to reduce friction losses.

TABLE 2.—Relative Costs, Based on the Air Requirements for the 1950 Plant.

Pressure.	Annual Cost of Compressing 150 000 Cu. Ft. of Free Air per Min.	Cost of Air Mains.		Annual Charges.
	Percentage of cost.	Percentage of cost.	Interest, percentage on cost, at 6%.	Percentage of cost of compressing air plus interest on pipe installation.
(1)	(2)	(3)	(4)	(5)
7 lb. 4 oz..	90+	174—	174—	93—
7 " 5 " ..	91—	168+	168—	93+
7 " 6 " ..	91+	161+	161+	94+
7 " 7 " ..	92+	155+	155+	95—
7 " 8 " ..	93+	149+	149+	95+
7 " 9 " ..	94+	143—	143—	96—
7 " 10 " ..	95—	137—	137—	96+
7 " 11 " ..	96—	131—	131—	97+
7 " 12 " ..	97—	125+	125—	98—
7 " 13 " ..	97+	118+	118+	98+
7 " 14 " ..	98—	112+	112+	99—
7 " 15 " ..	99+	107+	107+	99+
8 " 00 " ..	100	100	100	100

The assumption has been made that increasing the size of the air-pipes will increase the cost of the installation at a rate theoretically proportional to each 1 oz. increment of reduced pressure drop. This assumption, however, is not correct, because of the fact that, in enlarging the air pipes to reduce friction losses, only those sizes which show comparatively large losses would be increased substantially, and those which show lesser comparative losses would be increased in lesser proportion.

Although the comparative values, shown in Column (5) of Table 2, would not be the same as those which would result from actual cost analyses of various air-pipe combinations, corresponding to the pressure differences shown in Column (1), the general tendency of the annual charges to decrease as the friction losses are decreased, would appear to hold strictly true.

In consideration of the foregoing indications relative to the economical necessity for maintaining friction losses in the air mains at a practical minimum, a careful study of all the conditions entering into and affecting the actual installation of an air-pipe system was made, with the result that the combination of sizes finally adopted appeared to suit best the conditions.

The friction losses were calculated along the line, *A-B-C-D-E*, on Fig. 3, which is a diagram of the main air-piping for the 1930 plant and possible 1950 extension. Point *A* is in the power house; Point *B* at the junction of the 66-in. and 60-in. pipes in the gallery, just north of the center of the plant; Point *C* at the southwest corner of the south gallery; Point *D* at the extreme end of main air-header in the south gallery; and Point *E* at the top

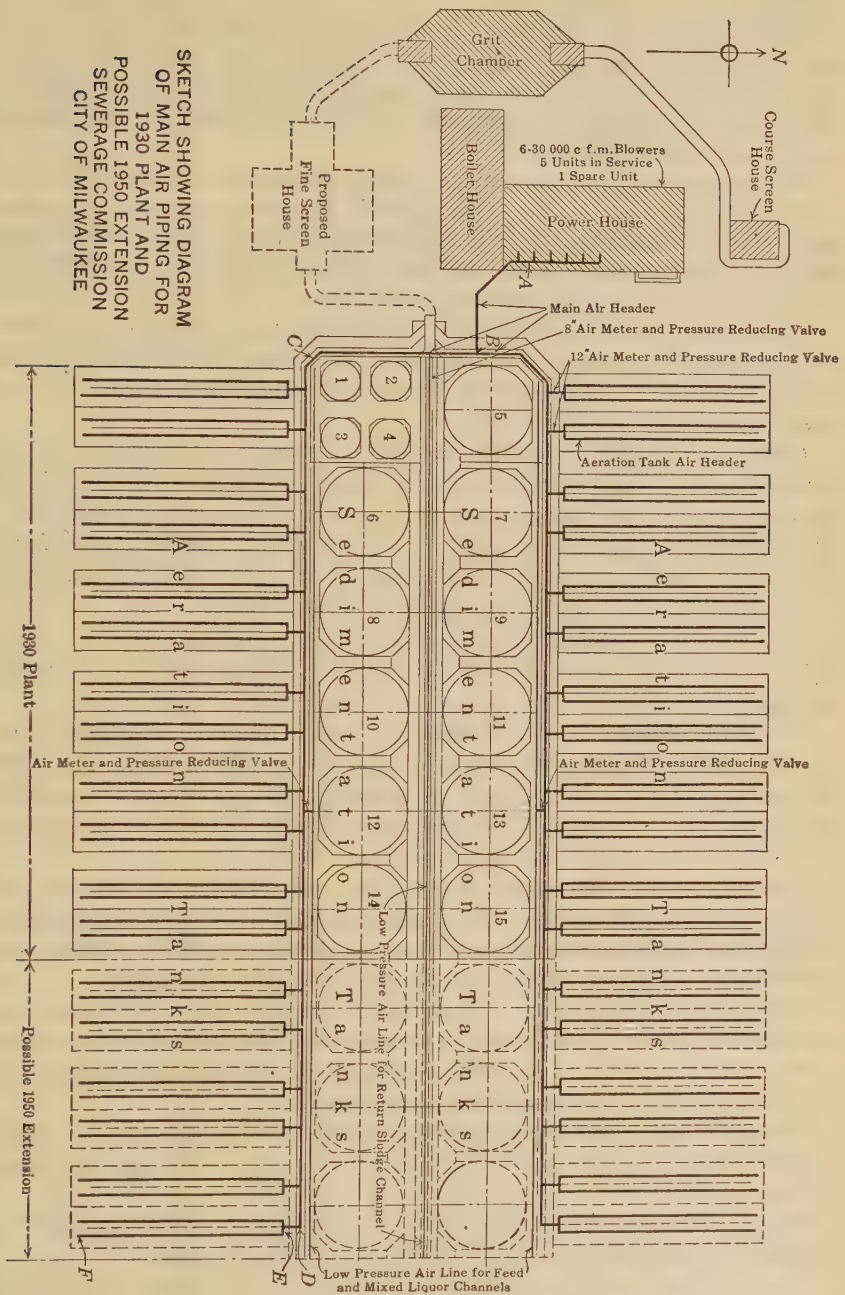

SKETCH SHOWING DIAGRAM OF MAIN AIR PIPING FOR 1930 PLANT AND POSSIBLE 1950 EXTENSION SEWERAGE COMMISSION CITY OF MILWAUKEE
Grit Chamber
N
Course Screen House
Boiler House
6-30 000 c f.m. Blowers 5 Units in Service 1 Spare Unit
Power House
Proposed Fine Screen House
A
Main Air Header
8" Air Meter and Pressure Reducing Valve
12" Air Meter and Pressure Reducing Valve
B
C
Aeration Tank Air Header
Aeration Tanks
Sedimentation Tanks
Air Meter and Pressure Reducing Valve
Air Meter and Pressure Reducing Valve
1930 Plant
Possible 1950 Extension
Low Pressure Air Line for Return Sludge Channel
Low Pressure Air Line for Feed and Mixed Liquor Channels
D
E
F

Fig. 3.

surface of the most remote diffuser plate in the far end of the most easterly aeration tank in the 1950 plant.

The total loss due to friction along this line is approximately 0.5 lb.; the fixed loss allowed for resistance through diffuser plates is 0.75 lb., which is simply safe, and the fixed losses occurring at the blowers is approximately 0.3 lb., making a total loss of approximately 1.55 lb., based on the estimated requirements for the 1950 plant.

The approximate pressure under which the blowers will possibly operate in 1950 is 8.06 lb. per sq. in. This figure is obtained by adding the pressure due to the 15-ft. depth of liquor in an aeration unit to the sum of the friction losses. It can be readily seen that the blower pressure for the 1930 plant will be somewhat less than 8.06 lb., possibly about 7.75 lb.

The pressure of the air supplied to the channel piping, as stated previously, will be proportioned, except for the small friction losses, to the depth of liquor in the channels. In all cases, these pressures will be considerably less than those in the aeration units.

Character of Pipe Required.—Precautions against the formation of rust which eventually may clog the diffuser plates, on the inside walls of pipes carrying air, is quite important in the design of an air-pipe system for the activated sludge process and should not be overlooked. The tendency for the pipe to rust is due principally to the presence of moisture in the compressed air, a condition which it appears is impossible to prevent.

The presence of moisture in the air is due to conditions of the atmosphere and also to the action of the water sprays on the air while it is passing through the air-washers.

No doubt, moisture will be present in the air mains in the form of condensate, resulting from differences in temperature between the compressed air, which will leave the blowers at probably 140° Fahr., and the outside atmosphere. This condition indicates the necessity for providing traps and blow-offs for removing condensate from the system.

Cast-iron pipe, Class A bell and spigot, was adopted for the main air-headers and for the headers in the aeration units. It was selected on account of its lasting qualities, rigidity, resistance to corrosion, and the ability of the lead joints to allow movement due to expansion and contraction.

Rigidity is referred to as an important factor with special reference to the larger sizes—from 36 to 66 in.—as compared with equal sizes of made-up commercial pipe of less substantial construction. All cast-iron pipe is to be well coated inside and out, with asphaltum paint.

All the sub-air headers and the small piping in the channels and aeration units are to be of galvanized wrought-iron pipe. All the fittings are likewise to be galvanized.

Diffuser Plates.—The diffuser plates to be used are a kiln-burned silica product and are known by the trade name of "filtros" plates.

These plates, which are 12 in. square and 1½ in. thick, are hard and porous and will not disintegrate or show signs of deterioration when they are immersed in sewage and subjected to the chemical actions taking place therein.

The specifications under which the filtros plates are being manufactured for the Sewerage Commission of Milwaukee provide that each plate, when dry, shall pass air at the rate of from 8.9 to 12.9 cu. ft. per min. under a 2-in. water pressure. The plates are to be placed in precast concrete containers. The setting of the containers and making the air-pipe connections thereto will not be done until the construction of the aeration units and channels is practically completed.

Notes and Data

A great deal may be said in connection with the details of the design for any large structure, especially one subjected to the influence of conditions which, by virtue of the environment of a fixed location, involve considerations relating to the study and application of hydro-static forces.

The conditions influencing and controlling the design of the structure for the disposal plant at Milwaukee do not differ, to any great extent, from those which have been encountered and studied elsewhere, and, although certain features of the design are based on data and analyses of conditions, probably not heretofore generally used, the design as a whole possesses no unusual features.

The shape and size of the tract for the sewage disposal works were determined in conjunction with the development of plans for the proposed harbor improvements. The decision arrived at relative to the arrangement of the disposal works as a whole was that the necessary structures for the cage screens, rack screens, fine screens, grit-chambers, blowers, generators, boilers, presses, and dryers, should be located on the fast land of the tract and that the aeration units and sedimentation tanks should be constructed on a site to be prepared in Lake Michigan adjacent to the fast land.

In accordance with this decision, a large coffer-dam was designed and constructed, and the enclosed area which is approximately 700 ft. by 900 ft., after having been dredged hydraulically to about Elevation —12.50, was dewatered.

The fact that all the structures would have to be placed on pile foundations was recognized from the beginning, and a series of tests were undertaken to determine the allowable bearing for the piles. A complete description of the pile tests, although containing much interesting data, is too lengthy to be presented herein. The final results, however, indicated that a bearing of 16 tons per pile would be sufficiently safe to use in connection with the design, and that figure, therefore, was adopted.

Conditions of Loading on Piles.—The condition of loading on which the spacing of piles was determined, took into account the total dead weight of the tanks and channels, and also the weight of the sewage contained in them when filled. The piles under the sedimentation tank bottoms are spaced 5 ft., center to center, both ways, and those under the aeration unit bottoms were spaced 5 ft. 6 in. in one direction and 4 ft. 7 in. in the other. In each case, the loading requires 1 pile for approximately each 25 sq. ft. of area, except under the walls where closer spacing is needed.

When the construction is completed, and the plant is placed in service, the ground-water level which, during construction, is being maintained lower

than the bottom of the site, by continuous pumping, will be allowed to seek its own level, which will probably be the same as the prevailing stage of water in Lake Michigan.

If the level of the water in Lake Michigan is Elevation 0.00, the uplifting force acting on the bottoms of the sedimentation tanks will be that due to a hydrostatic head of 13 ft., or about 812 lb. per sq. ft., and that acting on the bottoms of the aeration units will be about 740 lb. per sq. ft., or that corresponding to a hydrostatic head of 11.83 ft.

In considering the uplifting force in connection with the design, full value was given to the intensity of this force, no allowance whatever being made for resistance due to percolation through the soil, or for the probable inability of the pressure to exert itself over the entire bottom of the tanks, which will be poured in place directly on the sandy bottom of the tract.

This uplifting force, of course, will be more than balanced by the weight of the tanks and channels which will be filled with sewage when the tanks are in service, and partly balanced by the weight of the structures alone when empty. It is the latter condition which must be met to insure the stability of the plant against conditions likely to be met in operation, which would require the emptying of the units.

Two methods of satisfying this condition were carefully considered: One to provide sufficient weight in the structure itself to balance the uplift, and the other, to provide a sufficiently rigid anchorage or bond between the pile-heads and the concrete and utilize the holding-down power of the piles.

For reasons of economy, the latter method appeared to be the more favorable, and a series of elaborate tests were conducted to determine, if possible, the best and most adaptable type of anchorage.

The tests, which were conducted by R. R. Lundahl, Assoc. M. Am. Soc. C. E., Division Engineer, indicated that a plain pile-head, stripped of the bark and embedded 10 in. in the concrete, would develop strength in bond, with an ample safety factor, sufficient to equal the holding down power of the pile, which was fixed, with safety, at 50% of the bearing value, or about 8 tons. The maximum uplift on the piles as spaced will not greatly exceed 6 tons per pile.

In view of the results of these tests, the designs provided for each pile-head to be embedded 10 in. in the concrete bottom, and the weight of the structures was based on the requirements for strength alone.

Design of Bottoms.—Flat slabs, reinforced in two directions, top and bottom, were used for the sedimentation tank and aeration unit bottoms. The loads used were those which have already been mentioned in connection with the bearing and uplift on the piles.

A section of slab, the length of each side being equal to twice the distance between pile centers, was used as the unit area on which to calculate the thickness of the concrete and the percentage of steel.

The selection of the dimensions for the slab, which dimensions were considered sufficiently substantial to warrant their use, was governed by considerations relative to the possibility of unequal settlement of the piles and the resulting tendency to set up uncertain conditions of strain in the bottoms,

which, if not adequately guarded against, might be conducive to ultimate rupture.

The steel reinforcement in the top of the slab is proportioned to the moment resulting from the hydrostatic uplift, and the negative moment over the supporting piles is due to the force of the downward loads. The steel in the bottom of the slab is proportioned to the downward loads acting independent of the uplift.

Design of Walls.—Main walls of the cantilever type, reinforced on both sides and supplied with 0.003% of temperature steel, were designed for the aeration units, sedimentation tanks, and channels.

Walls subject to the same water pressure on both sides were proportioned and reinforced to withstand the total load from either side. Walls subject to a combined earth and water load on one side and a water load on the other side, were treated in a similar manner.

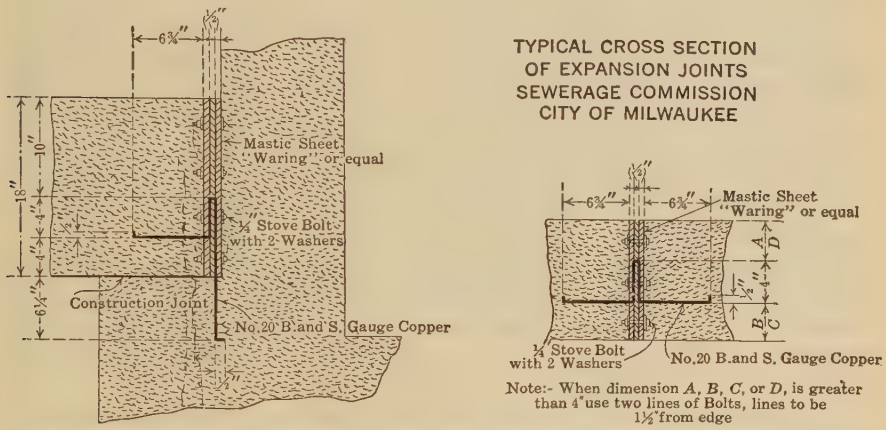


FIG. 4.

The loading referred to as combined earth and water is that due to back-fill in contact with outside walls and to the hydrostatic condition of the back-fill which is assumed to be completely saturated below zero elevation. The horizontal unit pressure exerted by the saturated back-fill is assumed to be about 100 lb. per sq. ft. for each foot of depth.

The tension allowed in the steel throughout the design was 16 000 lb. per sq. in., and the maximum compression allowed in the concrete was 650 lb. per sq. in.

Expansion Joints.—Expansion joints of the general type shown as Sections *S-S* and *T-T* in Fig. 4, were incorporated in the design. Where used, these joints completely separate the adjacent parts of the structure.

Each entire section of the plant between the expansion joints is securely tied together with reinforcing rods and, from a structural standpoint, is a self-contained unit. The copper sheet is to be stitch-riveted and soldered at joints, corners, and intersections, in order to secure water-tightness.

Galleries.—The operating galleries will be constructed of brick, with reinforced concrete and composition roofing and steel roof beams.

The exterior treatment will harmonize with the general architectural scheme adopted for all the buildings which will have been constructed when the entire plant is completed.

The design contemplates the use of steel doors and windows throughout. Subway grating, 1 in. deep, of "Irving" Type G, in connection with concrete slabs, will be provided for walk-ways and covers throughout.

General

The "thickeners", or concentrators, with which all of the sedimentation tanks are to be equipped, will be supported from steel trusses spanning the tanks. The reduction gearing, motors, etc., for driving the apparatus, will be mounted on the top chords of the trusses and will be protected from the elements by housings of hy-rib construction.

The starting and stopping switches will be mounted on the walls inside the center gallery and near the sludge outlet pipes, thus placing the complete sludge-control mechanism for each unit conveniently close to the unit itself.

A 4-in. water main carrying city pressure is provided for each of the three galleries. These mains which will be suspended from the roof beams, are to be fitted with drop pipes and hose connections $2\frac{1}{2}$ in. in diameter, opposite each sedimentation tank and each aeration-tank battery, for the purpose of providing facilities for flushing the tanks, channels, and drains.

The present filtros-plate containers and the container separators are being manufactured at the rate of about 60 pieces per day, by the Sewerage Commission, in its own casting plant which was constructed especially for this purpose. The plant is completely equipped with the necessary steel forms, cranes, tram-cars, mixing plant, and steam room.

The manufacture of the containers and separators, particularly the containers, is work requiring extreme care, and the product being turned out of the plant is first-class in every respect.

Conclusion

The work has been directed and developed under the direction of T. Chalkley Hatton, M. Am. Soc. C. E., Chief Engineer of the Sewerage Commission, and for several years past Harrison P. Eddy, M. Am. Soc. C. E., has been employed by the Commission in the capacity of Consulting Engineer.

Those of the Sewerage Commission staff, beside the writer, who have been associated intimately with the development of the process from its inception, and also with the more recent work of preparing final designs, are James L. Ferebee, M. Am. Soc. C. E., Principal Assistant Engineer; William R. Copeland, Affiliate, Am. Soc. C. E., Chief Chemist; A. Lawrie Kurtz, Assoc. M. Am. Soc. C. E., Division Engineer and Designer; Henry M. Reisig, Assistant Chief Chemist; Anthony J. Magerl, Architectural Engineer; H. Erskine Nicol, Assoc. M. Am. Soc. C. E., Senior Engineer; and M. Bert Langeler, Structural Engineer and Designer.

DISCUSSION

HARRISON P. EDDY,* M. AM. SOC. C. E. (by letter).—This admirable paper is of particular value at this time, because it deals with a new and most interesting process of sewage treatment, and also because it describes features of a plant which, when completed, will be one of the largest in the United States for the treatment of sewage. As the paper deals essentially with the details and principles of design, it may be of interest to state briefly the reasons for adopting some of the features of the plant and for these particular designs.

At first thought, one might question the need of preparatory treatment of the sewage in grit-chambers, and, particularly, by fine screens. To understand the reasons for such treatment, it is necessary to have clearly in mind the process going on in the aeration tanks. In these tanks, a large quantity of air is introduced under pressure, slightly in excess of the hydrostatic head of the overlying water, for the dual purpose of supplying sufficient oxygen to promote ærobic bacterial processes and of mechanically agitating the sewage. Sufficient agitation is required to cause the sludge floc to be swept back and forth, so that it may come into intimate contact with all the finely divided suspended solids and colloidal matter, in order that such materials may be absorbed by the floc and the clarification thus produced may be complete. It is equally important, however, that the agitation shall not be so vigorous as to break up the fragile floc and cause it to return to its former colloidal state, and thus defeat the object of the treatment. Generally speaking, also, the more moderate the agitation, the less air and power required, and, therefore, the greater the economy.

Exhaustive experiments at Milwaukee, Wis., and elsewhere, have demonstrated that the heavy and coarse solids tend to settle and remain on the bottom of the tank, notwithstanding the introduction of a substantial quantity of air, and that a greater quantity of air is required to hold such solids in suspension than would be necessary if they were removed and only the finer material was allowed to enter the tanks. Unless the coarse solids can be kept in suspension, they will remain in the tanks and form deposits on the plates. Even when the Milwaukee sewage was passed through a screen having openings of $\frac{1}{8}$ in. or less, in width, it was found that such deposits were formed when the volume of air used was reduced below about 1 cu. ft. per min., with a plate-tank ratio of 1 : 4.4. It was decided, therefore, that economy and efficiency of treatment required the use of grit-chambers and fine screens preparatory to aeration.

Whether there is an additional economy in fine screening on account of a reduction in the air requirements of the sewage when devoid of its coarser suspended solids, is a mooted question. That the removal of suspended solids, if carried to a sufficient degree, would render the sewage more easily treated, cannot be doubted, but whether fine screens can accomplish a measurable effect in this direction, does not appear to have been proven.

* Cons. Engr. (Metcalf and Eddy), Boston, Mass.

The comparative efficiency of treatment in tanks 10 and 15 ft. in depth, was made the subject of careful and prolonged study at the Milwaukee Testing Station. Tests carried out with the greatest care, under the personal direction of William R. Copeland, Affiliate, Am. Soc. C. E., Chief Chemist, showed that, taking all the facts into consideration, it appeared that, for the same quantity of free air per gallon of sewage treated, there was little difference in the work accomplished by the 10-ft. and the 15-ft. tanks. The 15-ft. tanks have produced an effluent of somewhat greater stability, whereas the 10-ft. tanks have accomplished a slightly greater reduction in suspended matter and bacteria. If one depth of tank has an advantage over the other in purification efficiency, it seems to be the 15-ft. depth.

An important consideration in the selection of depth of tank is the power required for compressing the air sufficiently to overcome the hydrostatic pressure of the several depths of sewage under consideration. In the cases of the 10 and 15-ft. depths, the latter would require 50% more power than the former, disregarding the effect of friction in pipe lines and porous plates. This saving will generally be offset, partly at least, by the increased cost of the larger area of tanks of the lesser depth. At Milwaukee, this is important, because of the cost of the bulkhead, the sheet steel enclosure, and the pile foundation. Another important factor there is the limited area available for the treatment plant, as it is felt that additional area could not be devoted to this purpose, on account of the requirements for harbor development.

Diffuser plates were early demonstrated to afford more satisfactory distribution and diffusion of air than any other method available, although it should be noted that there has been some conflicting evidence on this point. Comparative tests of plate-tank ratios of, approximatively, 1 : 4.4 and 1 : 6, were conducted for a period of four months at Milwaukee. The tests indicate that there is a slight advantage in the larger proportion of diffuser-plate area. This slight advantage was the reason for the adoption of the larger ratio in the design of the plant.

The design adopted for the sedimentation tanks has given rise to more misgivings, on the part of engineers connected with the work, than any other feature described by the author. There is no precedent in this process for tanks of this size, and grave doubt exists in the minds of some as to the applicability of the experimental data to tanks differing so widely in size. There does not appear to be serious uncertainty as to the operation of the tanks as sedimentation units, although the larger the surface area, the greater will be the effect of the wind in causing the floc to remain in suspension and pass out in the effluent. The behavior of the sludge accumulation, however, is a different matter. The success of the tanks, in fact, the efficacy of the entire process, depends in large measure on keeping the sludge as fresh as possible. It is important, therefore, to remove all the sludge promptly, otherwise it will tend to change in character, losing its "activity". If all the solids were to settle promptly to the floor of the tank, and the scrapers, or squeegees, could be relied on to push effectively the sludge to the center of the tank, the length of time in travel probably would not be long

enough to permit material impairment of the sludge. This, however, does not seem to be the case. Most of the floc moves slowly in its vertical passage toward the bottom of the tank, and enters a horizontal trend of flow long before it reaches the bottom. In certain small-scale experiments, the writer found that the solid particles after settling to a substantial depth moved toward the center in an almost horizontal direction. If this tendency extends to so long a distance, that is, about 50 ft., the large tanks adopted should not present serious operating difficulties. However, if the sludge does not escape promptly and completely, it may be necessary to empty and wash out the tanks at comparatively short intervals. Although such procedure does not offer prohibitive difficulties, it would introduce an annoying feature of operation.

The depth of the settling tanks—15 ft.—was decided on for the purpose of affording sufficient opportunity for fluctuations in the depth of sludge accumulation or "blanket", which varies greatly from hour to hour during the day. This variation appears to be caused primarily by the changes in quantity and quality of the sewage being treated, the quantity and density of return sludge being maintained as nearly constant as possible. Obviously, the smaller volume of weak sewage received at night does not yield as large a quantity of sludge as the greater volume of stronger day sewage. Hence, the sludge accumulation in the settling tanks—and to a less extent the sludge burden in the aeration tanks as well—varies greatly, even though the withdrawal of sludge is uniform as to volume and density. It is necessary, therefore, to provide ample storage capacity in the settling tanks to prevent the sludge from accumulating to a depth sufficient to force the fine surface sludge blanket to pass out in the effluent. The fluctuations in depth of sludge of different densities during a typical day are shown by Table 3.

TABLE 3.

Density of sludge,* percentage.	Minimum depth, in feet, at 6 A. M. ±	Maximum depth, in feet, at 6 P. M. ±
1	6	12
10	3.5	9.5
90	2	5

* The volume of sludge remaining at the bottom of the test sample after standing quiescent in a glass cylinder, 2¼ in. in diameter and 16 in. high.

It is evident that there was some lag between changes in volume and character of sewage and those in depth of sludge accumulation, the minimum depth of sludge occurring at about 6 A. M. and the maximum at about 6 P. M., whereas the corresponding changes in the sewage occurred several hours earlier.

These measurements illustrate the normal fluctuations, although the depth can be controlled, within limits at least, by regulating the rate of drawing surplus or "waste" sludge—the sludge in excess of that required for treatment of incoming sewage.

Comparative tests were not made to determine whether there was particular merit in depth as contrasted with volume of sludge storage space in the settling tanks, although, theoretically, there would appear to be some advantage in a shallower tank of equal volume. In the absence of definite information on this subject, and in view of the rapid and large fluctuation in depth of sludge within the experimental tanks, it was decided to build the settling tanks 15 ft. in depth.

ARTHUR L. SHAW,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has had occasion to study some features of the plans for the sewage purification plant, at Milwaukee, Wis., in connection with the work of Harrison P. Eddy, M. Am. Soc. C. E., as Consulting Engineer to the Commission, and has been much interested in the author's comprehensive outline of the net results, in structural terms, of the exhaustive studies on which the final designs of this notable purification plant have been based. (The word, "purification", is used advisedly, as it is believed to be more appropriate in the present instance than the word, "disposal". A process so scientific, and productive of such a high degree of purification, seems worthy of the former term, reserving the latter for use in the general sense, or for those methods which are less complete in treatment.) As a résumé of the basic data for the structural and hydraulic design of this plant, the author's paper will be of value and assistance to many designers who may encounter similar problems.

The designer of the Milwaukee works has been required to reconcile many conflicts between bio-chemical requirements and hydraulic and structural limitations and, although the practical success of the measures adopted will not be proven until after the plant shall have been placed in operation, he deserves much credit for the patience and ingenuity which his plans reflect.

Almost without exception, precedent points to the use of the air-lift for raising the settled activated sludge for re-circulation. The decision to use centrifugal pumps in the Milwaukee Plant for this purpose, is, therefore, a courageous one; and the effect of these pumps on the operating efficiency of the plant will be observed with interest. If the sludge floc is measurably broken up in passage through the pump runner, it is not unlikely that an appreciable part of the time of detention in the aeration tanks, after the return sludge has been added to the incoming sewage, may be consumed in the restoration of floc, before the sweeping action of the activated sludge on the colloids of the sewage can become fully effective. It is probable, also, that the pressing of pumped sludge may present some added problems, especially when pressing is undertaken without re-aeration and consequent opportunity for re-flocculation. The prohibitive cost of raising large quantities of return sludge continuously by the relatively inefficient air-lift has influenced the selection of the more economical method of pumping; and the experiment would appear to be worth the trial, particularly as a return to the air-lift method could be effected with little difficulty, should it prove necessary.

In discussing the rate of returning sludge, the author has somewhat overemphasized his objection to proportioning the rate of returning to the rate of sewage flow when he states that:

* Designing Engr., Metcalf and Eddy, Boston, Mass.

"Owing to the 'lag' through the plant, due to the large capacity of the tanks, it is questionable whether the synchronous method of operation could be controlled so as to approximate even the conditions of operation which, at first thought, appear to be the most desirable."

As a matter of fact, barring the slight effect of storage due to the small change in head on the outlet weirs, any change in the rate of flow of liquor entering a tank would practically be instantly felt in a corresponding change in the rate of discharge from the outlet end. The prism of liquor in the tank would function as an incompressible piston, any change in the rate of motion at one end being immediately apparent at the other end. The effect of large tanks and long periods of detention in causing "lag" or the smoothing out of sharp fluctuations in the rates of incoming flow is thus limited to that accomplished by the capacity for storage within the range between high and low-water levels in the tanks. This storage capacity, in the present instance, is only a small proportion of the total tank capacity, so that its effect is correspondingly insignificant.

A more important factor in the rejection of the synchronous method of returning, is the impracticability of meeting all the variables and uncertainties which enter into such a method. The character of sewage will change, as well as its rate of flow; the density of sludge is subject to wide variation at various seasons and even during any one day, and it is not only troublesome to determine the quantity of sludge of a given density necessary ideally to treat sewage of the character flowing at a given time, but there is no certainty that a sufficient quantity of such a sludge will be available at that time.

The decision to return sludge at a constant rate is thus based on sound principles and is borne out by the general success of this method at the demonstration plant, but it is to be noted that the channel and pump capacities provide for great flexibility in this respect. This is a wise provision, for it can be foreseen that conditions will arise, which may require radical departure from a fixed schedule of operation. For instance, when conditions are unfavorable, and a very light sludge is being produced, the sludge blanket in the sedimentation tanks will rise and may threaten to pass out over the effluent weirs unless the rate of sludge drawing is increased to prevent it. Uncertainties in this respect are enhanced by the adoption of sedimentation tanks of dimensions far beyond those of any experimental tanks.

The selection of Elevation + 3.00 as the maximum stage of Lake Michigan for which to provide, recalls the results of an analysis of the records of the recording gauge maintained by the U. S. Lake Survey at Milwaukee since 1903, which indicates that the occurrence of lake levels at or above this elevation once in about thirteen years is not improbable. Like other large inland bodies of water, Lake Michigan is subject to tide-like fluctuations or "seiches", which, at times, show marked rise and fall with respect to mean conditions within a few hours. Should the maximum recorded surge or rise above the mean occur simultaneously with a recurrence of the maximum recorded monthly mean, a lake level of Elevation + 3.28 would result. Such an occurrence, however, would be of brief duration, and provision for Elevation + 3 000 seems adequate in the present case.

There is lack of supporting data for the selection of Kutter's $n = 0.020$ in aerated channels. Its adoption is purely a matter of judgment, and it is to be hoped that some agency will undertake to study experimentally the friction losses in the aerated channels at Milwaukee when the plant shall have been placed in operation, so that, in the future, designers may have the benefit of data on this subject. Observations of the losses through the large sluice-gates of this project, operating as they do under low heads, would also supply information of which there is now a paucity.

A further field for research will be found in the air mains, which will offer opportunity for study of friction losses at relatively low pressures, particularly in bends, valves, meters, and other fittings, the present empirical allowances for which by various authorities are widely variant. Small losses, unanticipated, will be evident at the coal pile in a compressor plant of enormous capacity and in continuous operation, such as is required at a large activated sludge plant. Reliable data on these losses, therefore, will be of great value to designers of future similar plants.

This paper on the sewage treatment plant at Milwaukee contains many novel and interesting features of design. Those interested in the design and operation of plants of this type will look forward to the publication, not only of the analytical results of operation, but also of any hydraulic or structural data which operation may bring forth.

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Paper No. 1495

CONSTRUCTION PROGRESS OF THE HETCH HETCHY WATER SUPPLY OF SAN FRANCISCO, CALIFORNIA

By M. M. O'Shaughnessy,* M. Am. Soc. C. E.

With Discussion by Messrs. Joel D. Justin, Allen Hazen, Fred A. Noetzli, C. R. Rankin, and M. M. O'Shaughnessy.

Synopsis

The Hetch Hetchy Water Supply Project is to make available to San Francisco, and the adjacent metropolitan area, 400 000 000 gal. of water daily, sufficient to satisfy the requirements of the district for a century to come. The incidental production of 200 000 hydro-electric horse power will make it also an important factor in the industrial progress of Central California.

The water is to be diverted from the Tuolumne River and some of its tributaries, in the Sierra Nevada Mountains, more than 150 miles from San Francisco on the aqueduct line. The project takes its name from its principal reservoir site, Hetch Hetchy Valley.

It is not intended in this paper to describe or discuss in detail the project as a whole, but to present a comprehensive view of the present stage of construction, making only such reference to the general scheme and matters of policy as appear necessary to an understanding of the development of the working program.

Historical

From its pioneer days, the City of San Francisco has been supplied with water by the Spring Valley Water Company from local sources, owned or controlled by that Corporation. The most distant works now in use are, along the conduit lines, about 50 miles from the city. The approaching inadequacy of this supply has been evident for many years, and it was early recognized that

* City Engr., San Francisco, Calif.

the City must inevitably bring additional water from some more distant and more prolific source which, eventually, will become the principal one, the local works assuming the position of an auxiliary. Nearly every important stream in Northern and Central California has been proposed at one time or another as a possible source of the desired additional supply, and many of them have been made the subject of detailed engineering studies and cost estimates.

As early as 1879, the Upper Tuolumne River, with Hetch Hetchy Valley as a reservoir site, was suggested. In 1901, the Hon. James D. Phelan, then Mayor of San Francisco, filed water appropriation notices on the Tuolumne and its tributary, Elcanor Creek, and applied to the Secretary of the Interior for reservoir rights of way covering Hetch Hetchy Valley and Lake Eleanor, both of which lie in Yosemite National Park.

Opposition immediately developed, instigated primarily by corporate interests hostile to municipal ownership of public utilities, or financially interested in rival water-supply projects, and for twelve years progress was arrested by a well organized campaign of antagonism, with hostility by Federal officials to any encroachment in the Yosemite National Park. It was decided at last that special legislation would be necessary to grant the city the privileges it sought. Such legislation was enacted by Congress in December, 1913.

In the meantime, the people of San Francisco had declared themselves almost unanimously in favor of the project by voting, in 1908, a $600 000 bond issue for the purchase of privately owned lands and water rights, and, in 1910, by voting a bond issue of $45 000 000 to finance the construction.

In the spring of 1914, road work was begun, and, in the following July, the construction of a railroad to Hetch Hetchy Valley was commenced and a sawmill built. From that time, construction work has been pushed forward steadily, although until recently progress has been slow, due to the same difficulties of financing and obtaining labor and materials as were experienced by all large construction projects after the outbreak of the World War. A further handicap lay in a provision of the City Charter preventing the sale of the City's bonds at less than par. This was recently amended to allow the 4½% bonds already authorized to be sold on a 5½% basis.

Scope of the Project

The Hetch Hetchy System was first planned to supply San Francisco alone. The present scheme, however, is much broader. The cities on the eastern shore of San Francisco Bay and those south of the city on the Peninsula are even more in need of an increased water supply than San Francisco. Therefore, the region west, east, and south of the Bay was considered as forming a Metropolitan Water District. The ultimate capacity for the Hetch Hetchy works was fixed at 400 000 000 gal. per day and takes into account the probable requirements of the District throughout the present century.

John R. Freeman, President, Am. Soc. C. E., assisted the representatives of the city in presenting their case before the Federal officials, and, in 1912, prepared preliminary plans and estimates, in some detail, for developing ultimately 400 000 000 gal. of water daily and transmitting it to the Metropolitan District. The project is now being carried out along the general lines

then suggested by Mr. Freeman, although with modifications as to a number of important features. An abstract of Mr. Freeman's report has been published.*

Important Changes in Policy from the Plans of 1912

The plans of 1912 contemplated an initial development of 240 000 000 gal. per day, the surplus over the needs of the Metropolitan District to be used for irrigation in the adjacent area. Certain irrigation districts of the San Joaquin Valley protested against such a diversion of water from the Sierra Nevada water-shed for the irrigation of other regions, so this feature of the project was abandoned, and the initial pipe lines were reduced to sizes adequate for the domestic water supply, but have not been finally determined, as definite plans for the formation of a Metropolitan Water District have not been developed.

Another important departure from the former plans is in the program for power development. The intention was that the construction of hydro-electric plants should follow the completion of the first aqueduct unit delivering Tuolumne River water to San Francisco. The local sources of the Spring Valley water supply, however, are capable of further development which will meet the needs of the city for at least ten years to come, making the immediate delivery of Hetch Hetchy water unnecessary. On the other hand, electric-power development is of vital importance to the industries of California, and for many years the market will absorb new power as fast as it can be made available. It is to the advantage of San Francisco, therefore, to begin as soon as possible to generate the power which is a by-product of the water development, so that the revenue from power sales can be used to pay interest and redemption charges on bonds, reducing the burden to be carried by the taxpayers on the water project. A further and equally important reason for concentrating energy on the completion of the power project is that the water will thus be put to beneficial use and the water rights of the city thereby guarded, long before an aqueduct to San Francisco could be finished.

General Program of Construction

The following general program of construction, therefore, has been adopted. The Hetch Hetchy Dam is to be built to about three-fourths of its ultimate height and thus develop about 60% of the ultimate reservoir capacity; the aqueduct between Early Intake and Moccasin Creek is to be completed, and the power plant on Moccasin Creek is to be constructed, in order to begin generating electric power at the earliest possible date. It is expected that the power plant will be in regular operation before the end of 1923.

A 23-mile section of the aqueduct is to be built between Alameda Creek and the Crystal Springs Reservoir of the Spring Valley System, and will probably be completed sometime in 1924. This will make available, without duplication of aqueducts, the additional Spring Valley water previously mentioned, and, later, will carry Hetch Hetchy water.

* *Engineering News*, December 26th, 1912, pp. 1207–1214.

Construction of the remaining parts of the aqueduct will follow in time to have Hetch Hetchy water ready for delivery when the demand has absorbed the additional available supply from Spring Valley sources, but will be deferred, of course, as long as prudence will permit, in order to minimize the financial burden.

Other units of the project will be built as required, to keep pace with increasing demands for water and power.

General Description of the Project

Maps.—The location and condensed profile of the Hetch Hetchy System, from the water-sheds to the receiving reservoir in San Francisco, are shown on Plate VIII. The region from the Hetch Hetchy and Lake Eleanor Reservoirs to the head-works of the main aqueduct is shown in detail in Plate IX, and the location of that part of the main aqueduct now under construction appears in Fig. 1.

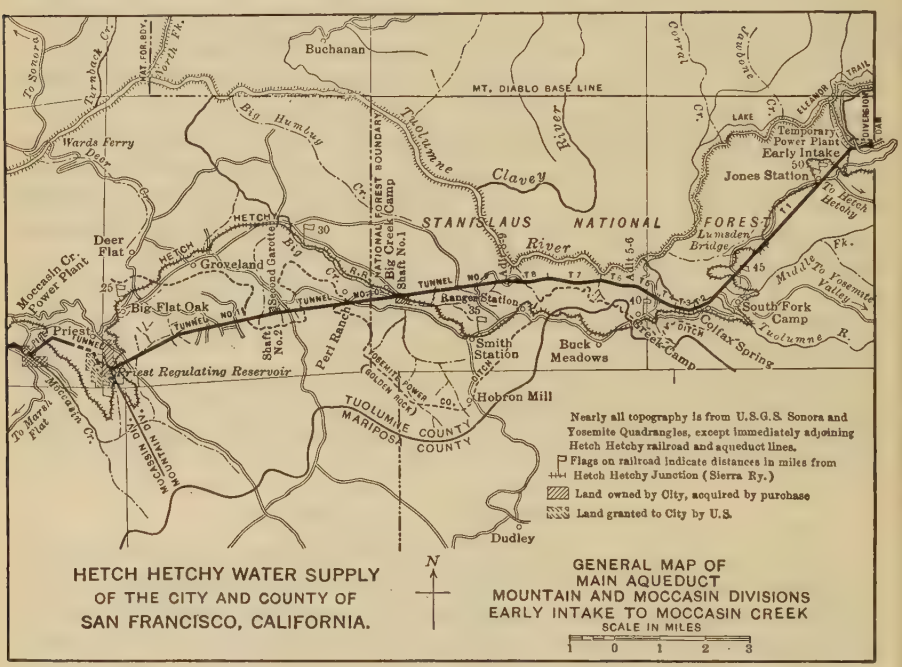
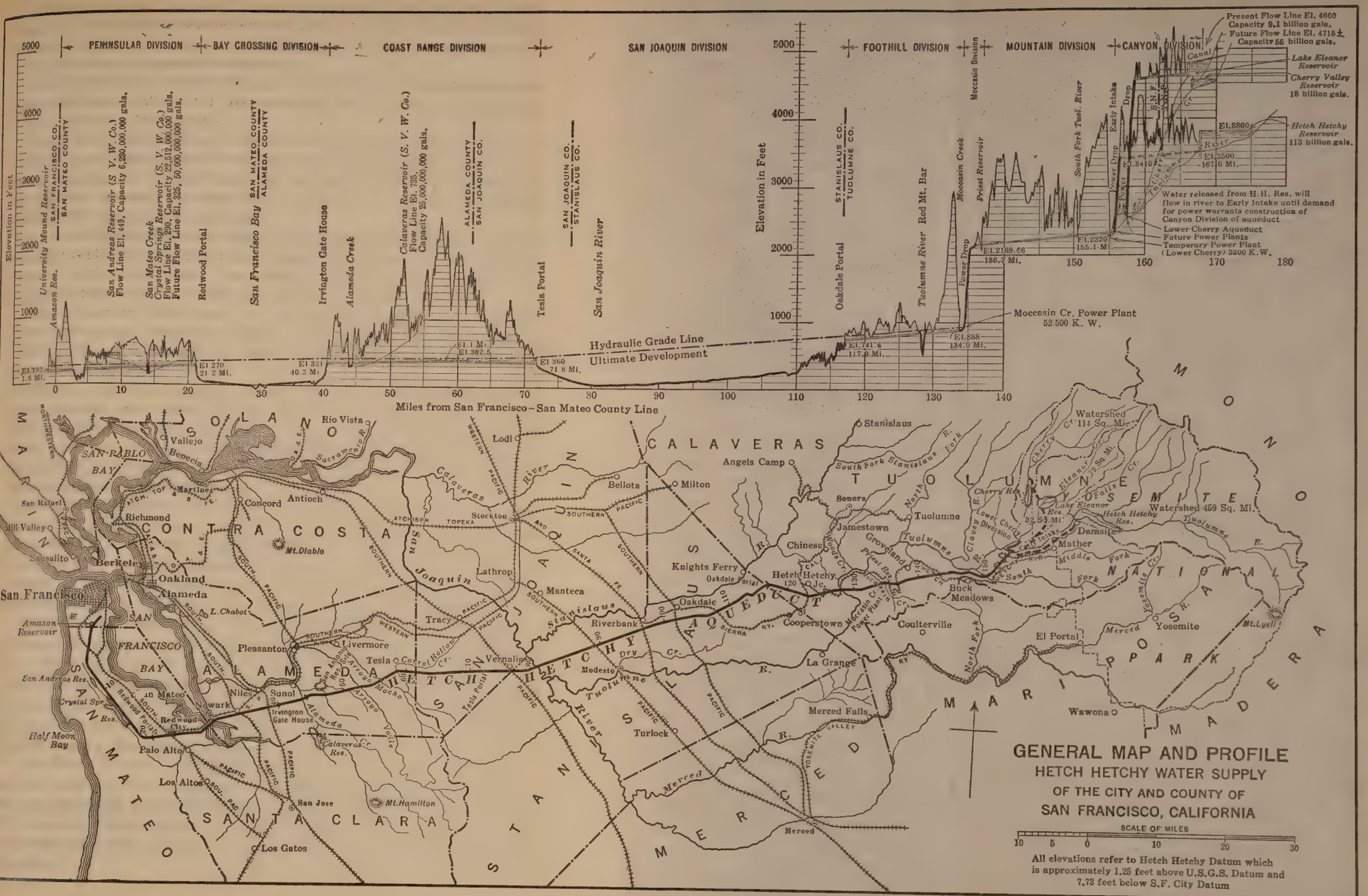


Fig. 1.

Water-Sheds.—The water-shed area tributary to the city's proposed impounding reservoirs on the head-waters of the Tuolumne River, covers 652 sq. miles in the high mountain region just west of the main ridge of the Sierra Nevada. Its elevation above sea level ranges from 3 500 ft. at Hetch Hetchy Dam to 13 090 ft. at Mount Lyell, 92% of the area being above Elevation 6 000. There are no permanent habitations, and the Congressional grant fixes sanitary regulations to prevent contamination of the water by summer camping parties.


PENINSULAR DIVISION
BAY CROSSING DIVISION
COAST RANGE DIVISION
SAN JOAQUIN DIVISION
FOOTHILL DIVISION
MOUNTAIN DIVISION
CANYON DIVISION
Elevation in Feet
Miles from San Francisco–San Mateo County Line
Hydraulic Grade Line
Ultimate Development
Moccasin Cr. Power Plant
52 500 K. W.
Present Flow Line El. 4660
Capacity 9.1 billion gals.
Future Flow Line El. 4715±
Capacity 55 billion gals.
Lake Eleanor Reservoir
Cherry Valley Reservoir
18 billion gals.
Hetch Hetchy Reservoir
113 billion gals.
Water released from H.H. Res. will flow in river to Early Intake until demand for power warrants construction of Canyon Division of aqueduct
Lower Cherry Aqueduct
Future Power Plants
Temporary Power Plant
(Lower Cherry) 3300 K.W.
San Andreas Reservoir (S. V. W. Co.)
Flow Line El. 449, Capacity 6,280,000,000 gals.
San Mateo Creek
Crystal Springs Reservoir (S. V. W. Co.)
Flow Line El. 290, Capacity 22,512,000,000 gals.
Future Flow Line El. 325, 50,000,000,000 gals.
Calaveras Reservoir (S. V. W. Co.)
Flow Line El. 735,
Capacity 25,000,000,000 gals.
Irvington Gate House
Alameda Creek
Redwood Portal
San Francisco Bay
Tesla Portal
San Joaquin River
Oakdale Portal
Tuolumne River
Red Mt. Bar
Moccasin Creek
Priest Reservoir
Moccasin Division
South Fork Tuol. River
Early Intake
Power Drop
Lower Drop
Tuolumne
Canal
River
Watershed 114 Sq. Mi.
Watershed 459 Sq. Mi.
32.3 Sq. Mi.
GENERAL MAP AND PROFILE
HETCH HETCHY WATER SUPPLY
OF THE CITY AND COUNTY OF
SAN FRANCISCO, CALIFORNIA
SCALE OF MILES
All elevations refer to Hetch Hetchy Datum which is approximately 1.25 feet above U.S.G.S. Datum and 7.73 feet below S.F. City Datum

The Hetch Hetchy and Lake Eleanor water-sheds, of 459 and 79 sq. miles, respectively, lie within Yosemite National Park. Yosemite Valley is 18 miles southeast of Hetch Hetchy, in the drainage basin of the Merced River, and is not affected by the water development.

Cherry Creek is to be made tributary to Lake Eleanor through a diversion canal and tunnel. (Plate IX.) Its water-shed lies in the Stanislaus National Forest, and covers 114 sq. miles.

Reservoirs.—Hetch Hetchy Reservoir, the elevation of which is 3 500 ft. at the original stream level, will have a total capacity, with a depth of 300 ft. at the dam, of 113 500 000 000 gal., or 348 000 acre-ft. In the initial development, however, the depth at the dam will be only 220 ft., storing 67 000 000 000 gal., or 206 000 acre-ft.

At Lake Eleanor, Elevation 4 600, a storage of 8 200 000 000 gal., or 25 300 acre-ft., has already been developed in connection with the construction power plant to be described later. The ultimate capacity at this site has been tentatively fixed at 54 000 000 000 gal., or 166 000 acre-ft.

Other reservoirs which may be developed later are: Cherry Valley, 18 500 000 000 gal.; Poopenaut Valley, 10 000 000 000 gal.; Lake Vernon, 16 600 000 000 gal.; Huckleberry Lake, 17 000 000 000 gal.; and Emigrant Lake, 4 600 000 000 gal.

The Aqueduct in the Sierra Nevada.—In the initial development, the water released from Hetch Hetchy Reservoir flows 12 miles down the Tuolumne River to a diversion dam at Early Intake. (Plate IX.) The fall in the river in this distance is 1 160 ft., and later will be utilized for power development, by means of a tunnel aqueduct in the granite canyon wall, on the south side of Tuolumne River.

The North Mountain Aqueduct.—In order to develop the full power possibilities of Lake Eleanor, an 8-mile aqueduct, with a capacity of 200 sec-ft., is to be constructed, leading from the Lake to a power drop at which a head of 2 100 ft. will be available. (Plate IX.) This power unit will be capable of generating continuously about 25 000 kw. The water will be discharged into the Tuolumne River above the Early Intake diversion.

The Mountain Division of the Aqueduct.—The aqueduct now under construction begins at Early Intake. (Fig. 1.) From the diversion dam at that point there will be ½ mile of canal with settling chambers, thence the water will be conducted through 18.3 miles of tunnel, discharging into the regulating forebay reservoir at Priest. This tunnel and all other tunnels of the main aqueduct are placed below the hydraulic grade line in order to operate under a light pressure head.

The Priest Reservoir will receive water at the constant rate of 400 000 000 gal. daily (619 cu. ft. per sec.), but will discharge at varying rates as required by the fluctuating load conditions at the power plant on Moccasin Creek. The reservoir will be formed by an earth and rock-fill dam 145 ft. high, with a concrete core wall. Its capacity will be 2 500 acre-ft., or 2 days' flow of the aqueduct.

An outlet tunnel, 1.3 miles long, will lead from this reservoir through a ridge to the head of the steel pressure pipe lines, each 4 700 ft. in length, which will supply the power plant on Moccasin Creek.

The Moccasin Creek Power Plant.—This power plant will have a 24-hour average capacity of 70 000 h. p. The maximum static head will be 1 315 ft. The initial installation will be three 20 000-kva. generating units, driven by double-overhung impulse water-wheels.

As the topography does not permit of the construction of forebay reservoirs above the other power plants of the System, these plants will be operated at a load factor of 100% in the ultimate development, and the entire system will be regulated at Moccasin Creek, more generating units being added to care for peak loads; hence, the necessity for the unusually large forebay capacity to be provided in the Priest Reservoir.

The general layout for this plant was determined in 1914 after consultation with a board composed of the following engineers, F. G. Baum, M. Am. Soc. C. E., Professor William F. Durand, and John D. Galloway, M. Am. Soc. C. E.

The Foot-Hill Division.—The aqueduct running west from Moccasin Creek (Plate VIII) will be constructed, when it becomes necessary to supplement the Spring Valley water supply with water from Hetch Hetchy, and will begin at the tail-race reservoir from the power plant on Moccasin Creek, at Elevation 900. This reservoir will receive the water from the plant and regulate the flow through the aqueduct, which is a tunnel and extends about 17 miles to Oakdale Portal, in the foot-hills east of the San Joaquin Valley.

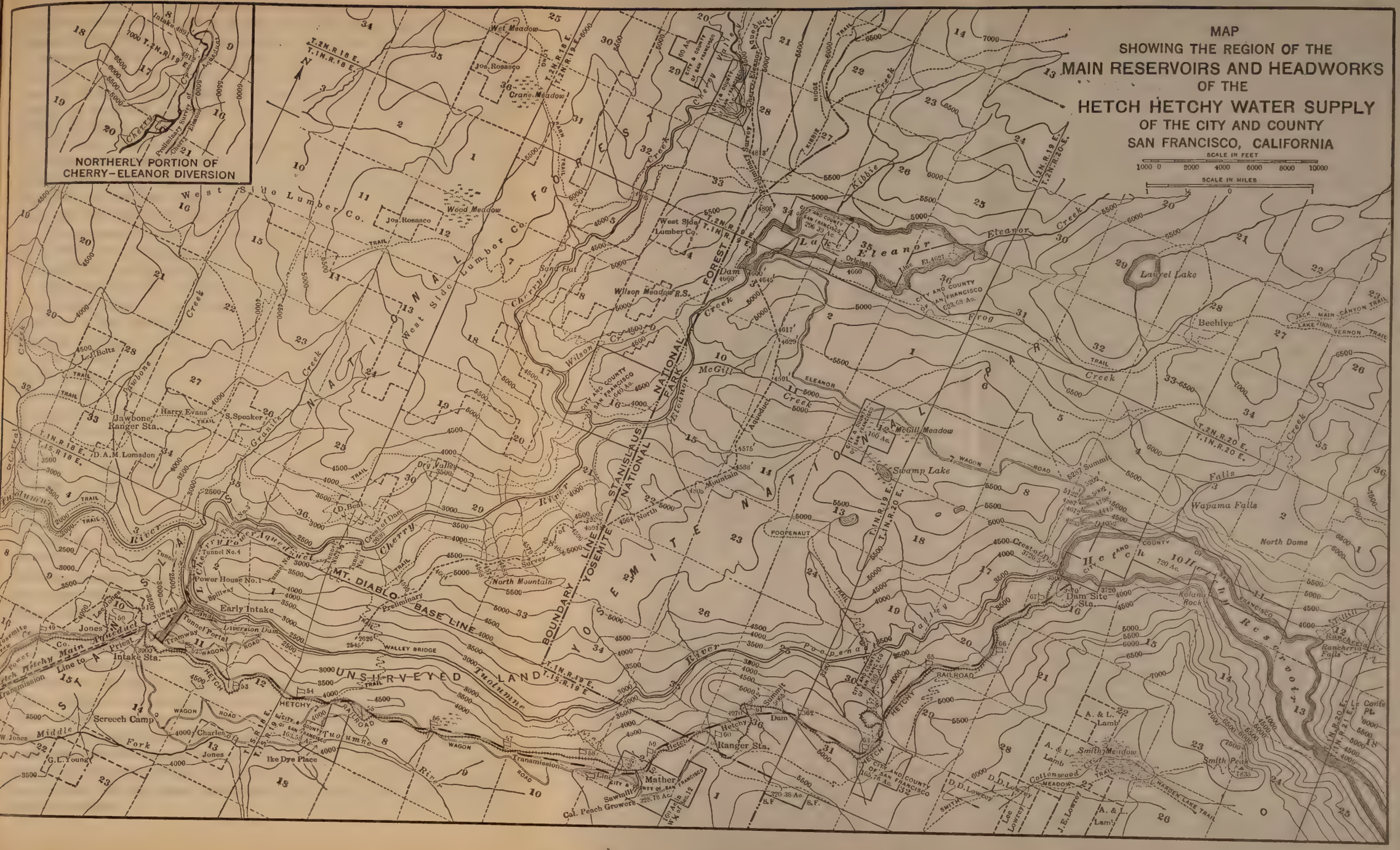
The tunnels so far mentioned are designed for a flow of 400 000 000 gal. per day, except the outlet tunnel from the Priest Reservoir, which is made larger in order to avoid excessive loss of head at the time of peak load on the power plant.

The San Joaquin Valley Pipe Line.—From Oakdale Portal, steel pipe lines will extend across the San Joaquin Valley, 45 miles to Tesla Portal. (Plate VIII.) The capacity of the first of these lines will depend on the development of the Metropolitan District, but is not likely to be less than 60 000 000 gal. per day. The final stage of the project will probably call for three lines nearly 7 ft. in diameter, each carrying 133 000 000 gal. per day. These pipes will be subjected to heads varying from 450 to 540 ft. for 70% of their length.

Coast Range Tunnel.—From Tesla Portal, a tunnel will penetrate the Coast Range, a distance of 31 miles, to Irvington Gate House, a few miles east of San Francisco Bay. Here, the aqueduct will divide to serve the three main divisions of the Metropolitan Area; the principal branch will extend westerly to the San Francisco Peninsula, another, northwesterly to the East Bay cities, and the third southward to San Jose.

At the crossing of Alameda Creek, a few miles east of Irvington Gate House, the waters of the near-by Calaveras Reservoir of the Spring Valley System will be taken into the Hetch Hetchy Aqueduct.

The Bay Crossing Pipe Line.—To supply the San Francisco Peninsula, an ultimate capacity of 200 000 000 gal. per day is planned.


MAP
SHOWING THE REGION OF THE
MAIN RESERVOIRS AND HEADWORKS
OF THE
HETCH HETCHY WATER SUPPLY
OF THE CITY AND COUNTY
SAN FRANCISCO, CALIFORNIA
SCALE IN FEET
1000 0 2000 4000 6000 8000 10000
SCALE IN MILES
1 ½ 0 1
NORTHERLY PORTION OF
CHERRY–ELEANOR DIVERSION
Cherry Creek
Eleanor Creek
Kibbie Ridge
Lake Eleanor
Cherry Lake
Laurel Lake
Beehive
Wapama Falls
North Dome
Rancheria Falls
Hetch Hetchy Reservoir
Poopenaut
Swamp Lake
McGill Meadow
Dry Valley
North Mountain
Early Intake
Power House No.1
Tunnel No.4
Tunnel No.5
Tunnel No.6
Tunnel No.7
Tunnel No.8
Tunnel No.9
Tunnel No.10
Tunnel No.11
Tunnel No.12
Tunnel No.13
Tunnel No.14
Tunnel No.15
Tunnel No.16
Tunnel No.17
Tunnel No.18
Tunnel No.19
Tunnel No.20
Tunnel No.21
Tunnel No.22
Tunnel No.23
Tunnel

Continuing west from Irvington, more than 19 miles of pipe line are required in crossing the low land adjacent to San Francisco Bay to the high ground on the Peninsula west of the Bay. The initial line will probably have a capacity of 30 000 000 gal. per day, and will be used first to carry the additional supply from the Spring Valley sources, for which purpose pumping must be resorted to, as the Spring Valley Conduit, which delivers water near Irvington, is 200 ft. below the hydraulic grade line of the proposed Hetch Hetchy Conduit.

The head on the pipe will range from 300 to 365 ft. for 17 miles of its length and where the navigable channel of the Bay is crossed, special construction must be used, the type of which, for the full development of the aqueduct, has not yet been determined. In the initial stage, submerged pipe with ball and socket joints will probably be used.

This connection ultimately will require three parallel pipe lines, each more than 6 ft. in diameter. The division terminates at a tunnel portal near Redwood City, called the Redwood Portal.

Connection to the Spring Valley System and to San Francisco.—The preliminary plans of the aqueduct from Redwood Portal to the receiving reservoir in San Francisco, a distance of 22 miles, call for a tunnel 10 ft. in diameter, broken at two places by siphons which total 2 miles in length.

The Crystal Springs Reservoir and two other reservoirs of the Spring Valley System lie near the aqueduct in this division. For the first stage of development, it is planned to construct about 3 miles of tunnel from Redwood Portal and establish a connection to Crystal Springs Reservoir, the capacity of which is 22 000 000 000 gal. The additional Spring Valley supply from Alameda Creek will then become available to San Francisco without further conduit construction being necessary for a few years, as the existing pipe lines of the Spring Valley System between Crystal Springs and San Francisco, with their booster pumps, are capable of furnishing more water than is now available in that System.

Receiving Reservoir in San Francisco.—The Hetch Hetchy Aqueduct will terminate in San Francisco at the proposed Amazon Receiving and Distributing Reservoir, which is to have a capacity of about 300 000 000 gal. Its high-water elevation will be 248 ft. above sea level. From this elevation, about half the supply can be distributed without pumping. Delivery by gravity at a higher elevation would unduly increase the cost of the aqueduct and would prevent the use of the Crystal Springs Reservoir in connection with the Hetch Hetchy supply.

Relation of the Spring Valley and Hetch Hetchy Systems.—It is the settled policy of the City Government to acquire all properties of the Spring Valley Water Company that are of permanent value for water supply purposes, and the plans for the Hetch Hetchy development provide for connections to co-ordinate the two systems and avoid the waste of duplication of structures. The distant supply would thus be combined with a local system which affords large storage for the waters from the distant source and which itself is a source of supply.

The proposition to purchase the local system has been made the subject of three elections, and has received an affirmative majority vote each time, but not the two-thirds majority necessary under the City Charter for consummation of purchase by approval of a bond issue.

Organization and Field Headquarters

The Hetch Hetchy water supply is under the jurisdiction of the Department of Public Works of San Francisco. The City Engineer is Chief Engineer of the Hetch Hetchy Project. His Chief Assistant directs all activities, and the headquarters engineering work is carried out by a branch of the City Engineer's office. In the field, two construction engineers report directly to the Chief Assistant Engineer; one is located at Groveland and the other at Hetch Hetchy.

The general headquarters of the construction work in the mountains are located at Groveland, 27 miles on the City's Railroad from Hetch Hetchy Junction and 41 miles from Hetch Hetchy Valley. The Construction Engineer at Groveland has charge of the aqueduct work and is also Superintendent of the Hetch Hetchy Railroad. The Construction Engineer at Hetch Hetchy supervises the work on the Hetch Hetchy Dam and Reservoir and at Lake Eleanor, and the operation of the Mather saw-mill.

Hospital Service.—The City has provided a completely equipped hospital located at Groveland. The contractors are required either to avail themselves of the city's medical service for their employees, paying the city a fixed amount per employee per month, or to provide similar facilities at their own expense.

Shops.—The City's machine shops and car repair shop at Groveland handle practically all repairs in connection with the railroad operation and aqueduct construction, including heavy locomotive work. Several complete railway cars have been constructed in these shops.

Description of Construction Work Completed and in Progress

Following the general program already outlined, all construction activities thus far undertaken have been concentrated in the Sierra Nevada region between Lake Eleanor and Hetch Hetchy Reservoirs and Hetch Hetchy Railroad Junction, a distance of 80 miles.

Immediately after the Hetch Hetchy grant was signed, work was begun to open up transportation facilities in the mountains. Pending the construction of a railroad, the existing roads were improved and relocated, and additional roads were built to Hetch Hetchy and Lake Eleanor, which were previously accessible only by horse trails.

To give access to Hetch Hetchy Valley, the 9 miles of railroad grade from Mather was rushed to completion in 1914, while surveys for the remainder of the line were still in progress. Most of the heavy hauling over the 55 miles of road from Chinese, the nearest railroad station, to Hetch Hetchy, was done under contract by motor trucks, at from 40 to 60 cents per ton-mile, the

rate depending mainly on the condition of the roads. For four months of the year, heavy hauling was prevented by the impassable condition of the dirt roads.

The Hetch Hetchy Railroad

All construction supplies are now transported to Oakdale (Plate VIII) on the Southern Pacific or Santa Fe Lines, and are there transferred to the Sierra Railway of California. Hetch Hetchy Junction, the initial point of the city's railroad, is 26 miles from Oakdale, on the Sierra Railway. The Hetch Hetchy Railroad extends 68 miles easterly from the junction to the construction camp at Hetch Hetchy Dam, known by the station name of Damsite. The air-line distance between the termini is about 40 miles.

The railroad serves the working points of the 30 miles of main aqueduct east of the Sierra Railway, the power development on Moccasin Creek, and the City's saw-mill, some directly, others through short spur-tracks, tramways, or motor-truck hauls.

The Hetch Hetchy Railroad was constructed under two contracts, one let in 1914, covering the grading of the 9 miles from Mather to Hetch Hetchy; the other was carried out in 1916 and 1917, covering all grading, track work, and other construction necessary to complete the railroad from Hetch Hetchy Junction to Damsite.

The Lower Cherry Power System

A temporary power plant located at Early Intake generates electric power for all the construction work from the Hetch Hetchy Dam to Moccasin Creek. Water is diverted from Cherry River into a conduit, 3½ miles long, which has a capacity of 200 sec-ft.

There are three horizontal Pelton-Francis turbines, each designed to operate at 720 rev. per min., and to develop 1 500 h. p., under a maximum head of 345.5 ft. The generators, which are direct connected to the turbines, are 2 300-volt, 3-phase, 60-cycle machines, each of a rated capacity of 1 000 kva., with direct-connected exciters.

Two transmission lines conduct the power to the construction work. One line extends 14.5 miles east to the Hetch Hetchy Dam, and the other 19 miles west along the main aqueduct line to Priest, with a 2-mile branch to Groveland.

To insure sufficient water to operate the plant over the dry season, when the natural flow of the Cherry River shrinks to almost nothing, the Lake Eleanor Dam, described subsequently, was built.

Hetch Hetchy Dam

The Hetch Hetchy Dam is being constructed in two installments. Some of the principal dimensions of the initial and the ultimate stages are given in Table 1. The initial dam is now under construction under a contract awarded in August, 1919, to the Utah Construction Company.

TABLE 1.—Statistics of Hetch Hetchy and Lake Eleanor Reservoirs.

	Hetch Hetchy Reservoir.		Lake Eleanor.	
	Initial.	Ultimate.	Present.	Ultimate.
Area of water-shed, in square miles	459	459	79	183*
Capacity of reservoir:				
In millions of gallons	67 000	113 500	9 100†	54 900†
In acre-feet	206 000	348 500	28 000	168 400
Water surface area, in square miles	2.5	3	1.5	2.2
Elevation of roadway on dam, in feet	3 726.5	3 812	4 661	4 785
Elevation of spillway crest, in feet	3 719.75	3 800	4 660‡	4 775
Length of reservoir, in miles	7.5	8	3.1	3.2
Width of reservoir, maximum, in miles	0.65	0.7	1.0	1.1
Width of reservoir, average, in miles	0.33	0.38	0.5	0.7
Depth of reservoir from spillway crest:				
Maximum, in feet	220	300	60†	175†
Average, in feet	129	179	29†	119†
Dam: Type of dam	Concrete, gravity section, arched in plan.		Reinforced concrete buttressed arch.	Rock-fill with concrete facing.
Length on crest, in feet	600	900	1 260	1 750
Height of crest above stream level, in feet	226 (Roadway)	312 (Roadway)	60	185 (Roadway)
Depth from stream level to bed-rock, at up-stream toe of dam, maximum, in feet	101	101	Stream bed is solid rock	
Depth from stream level to lowest point in foundation, in feet	118	118	.....	.....
Height of dam, above bed-rock at toe, maximum, in feet	327	413	61	.....
Height of dam above lowest point in foundation, in feet	344	430	70	.....
Width at crest, in feet	15	25	.....	.....
Width at base, maximum, in feet	298	298	.....	.....
Volume of masonry, in cubic yards	375 000	625 000	11 640	.....
Type of spillway	Siphon.	Channel around end of dam.	Overflow.	Channel around end of dam.

* Includes Cherry Creek water-shed above proposed diversion.
† Lake Eleanor depths and capacities do not include that part of the original lake which is not available for draft.
‡ With flash-boards in place; 4 655 ft. without flash-boards.

The addition necessary to bring the dam up to its ultimate height and thickness will be made when the demands for water or for electric power justify it. The present construction includes all work below stream level necessary for the ultimate structure.

Design.—The dam is to be of the gravity type, arched in plan, with the radius of the up-stream face at the crest 700 ft. The cross-section, at the maximum section, is shown in Fig. 2.

Most of the dam will be of 1 : 3 : 6 concrete embedded with large stones. For the concrete against the foundation, the cut-off trench, the up-stream face, and the down-stream face in the spillway section, a 1 : 2½ : 5 mixture is to be used to give greater impermeability.

The limiting working stresses in the design were as follows:

Pressure normal to joint, up-stream toe:
Reservoir empty .....25 tons per sq. ft.
Reservoir full .....16 tons per sq. ft.
Maximum pressure in section, under worst combination of conditions and lasting for comparatively short periods of time.....27.5 tons per sq. ft.

No tension was permitted in the concrete. On each joint, upward water pressure was considered as acting at the up-stream face with an intensity of two-thirds the total hydrostatic head and diminishing uniformly to zero at the down-stream face. On the foundation joint, an upward pressure of two-thirds the hydrostatic head due to the back-water was also considered.

The dam is to be penetrated by a system of inspection tunnels, inspection wells, and drainage wells. The inspection wells will be lined with dense concrete blocks and equipped with ladders. The drainage wells will be in porous concrete blocks and will be 15 in. square.

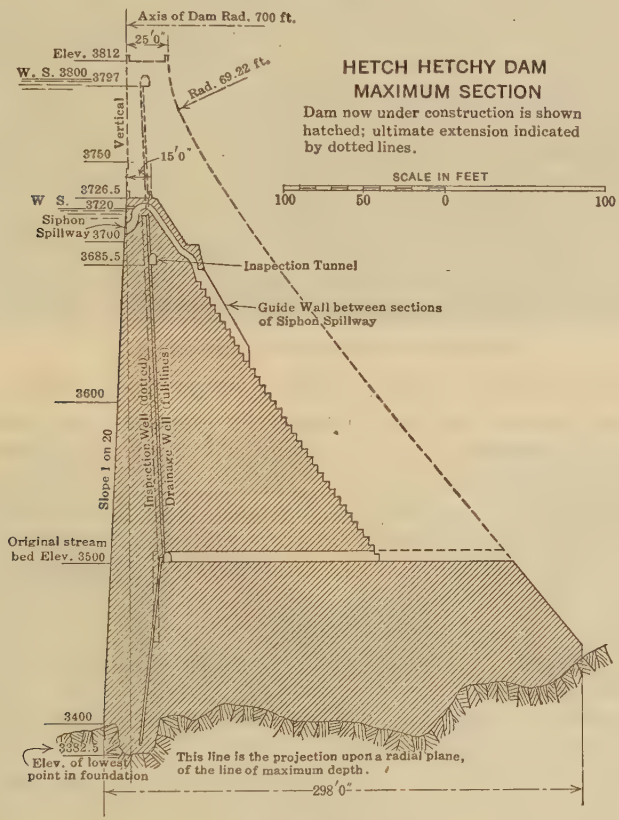


FIG. 2.

Radial contraction joints, sealed by bent copper water-stops, are provided at intervals of about 100 ft., measured along the up-stream face. Each contraction joint bisects an inspection well. The crest of the dam will be used as a roadway, in both the initial and ultimate developments. The stresses in the typical dam section were checked in 1917 by C. D. Marx, Past-President, Am. Soc. C. E., and Charles B. Wing, M. Am. Soc. C. E.

Spillway.—The ultimate dam will have a spillway of the weir type, with a channel to carry the waste water around one end of the dam. A siphon spillway has been adopted for the initial dam, and will be in eighteen sections,

staggered in elevation to obviate vacuum effects, with a total length of clear openings of 180 ft. 6 in. Each section will be 8 ft. high at the entrance, tapering to 4 ft. at the crest of the siphon, and will have two air vents, each 12 by 24 in.

Outlet System.—There will be twelve outlet conduits, six of which will be each 5 ft. in diameter, and arranged in pairs at three levels. These conduits will discharge water (in quantities up to 3 000 sec-ft.) which under certain conditions of river flow is permitted to pass the dam for the use of irrigation districts in the lower reaches of the Tuolumne River. Each of the other six outlets will be 3 ft. 6 in. in diameter, in two groups of three, and will discharge the water for the city supply. The smaller outlets will ultimately discharge directly into the aqueduct tunnel to be constructed in connection with the future power development at Early Intake, whereas the water for irrigation may flow directly down the bed of the river.

The discharge in each conduit will be regulated by balanced needle-valves with the Larner-Johnson type of control. At the entrance to the conduit, there will be a slide-gate with hydraulic cylinder operation. A heavy reinforced concrete shutter may be lowered through a slot in the wall of the outlet structure to close the opening on the up-stream side of the slide-gate, thus providing access to the gate for inspection or repairs.

Construction of Hetch Hetchy Dam

Excavation to the bed-rock foundation of the Hetch Hetchy Dam was completed in August, 1921, and immediately afterward the pouring of the concrete was commenced. It is the intention to pour nearly 400 000 cu. yd. of concrete in 16 months.

Stream Control.—The maximum measured flood flow of the Tuolumne River at the dam site, during the 10 years for which records are available, was 12 700 sec-ft., and the stream-control works were planned for approximately that quantity of water.

The river is by-passed around the dam site through a tunnel, 23 ft. wide, 25 ft. high, and 900 ft. long. The lower group of three 3 ft. 6-in. outlet conduits will be placed later in this tunnel.

The diversion dam is a rock-filled crib, 321 ft. long, of 12 by 12-in. timbers. The up-stream face is sheathed with a double layer of 2-in. plank, with a layer of tarred burlap between. A maximum head of 13 ft. over the top of the tunnel was provided to dispose of the flood waters. This structure was designed by J. Q. Barlow, M. Am. Soc. C. E., Consulting Engineer for the Utah Construction Company.

A concrete dam was built just up stream from the lower portal of the diversion tunnel, to prevent water discharged through the tunnel from backing up into the dam site. This dam was founded on boulders and coarse gravel, and is 51 ft. long on top. Its foundation was grouted to secure watertightness.

Excavation for Foundations.—A steam shovel was used for the greater part of the excavation in the dam site, loading directly into dump cars. (Fig. 3.) When the space became too cramped to permit steam-shovel operation,

Fig. 3.—Excavating in Hetch Hetchy Dam Site at Stream-Bed Level. Crib Diversion Dam in Background.

Fig. 4.—Hetch Hetchy Dam View Looking Down Stream. Excavation Practically Completed.

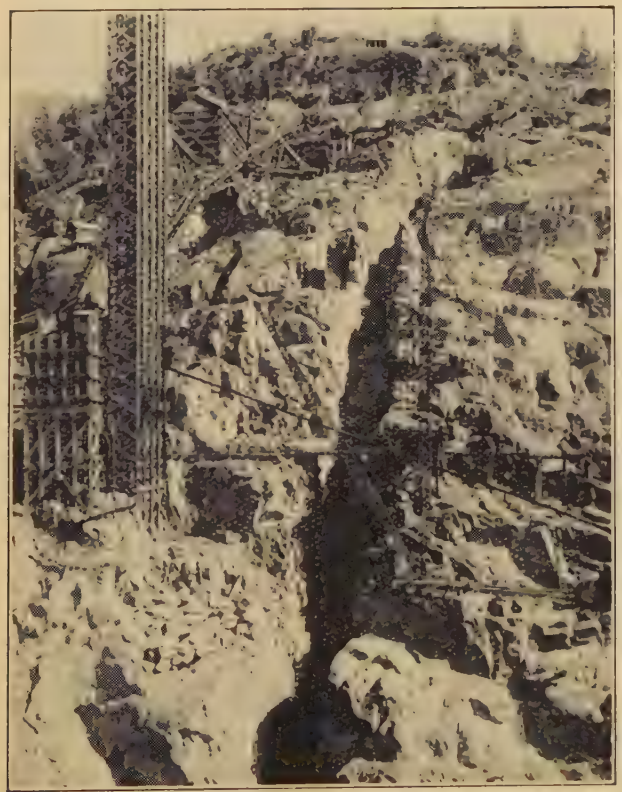


FIG. 5.—SOUTH ABUTMENT, HETCH HETCHY DAM, AFTER COMPLETION OF EXCAVATION.

FIG. 6.—CONCRETE PLANT, HETCH HETCHY DAM CONSTRUCTION.

the final 20 000 yd. was loaded into the skips by hand. The skips were emptied into trains running on trestles along the north and south walls of the gorge. (Fig. 4.) The total excavation for foundations was 165 000 cu. yd., of which 100 000 cu. yd. was solid rock. The trains dumped the material on the floor of the valley, and much of the dumped rock will be used for concrete aggregate. Large rock fragments and boulders were placed by derricks on convenient corners and ledges for storage until required for plums in the Cyclopean masonry. The lowest point in the foundation is 113 ft. below the original stream bed.

Nearly all the surface rock in the foundation above stream level was removed, leaving a newly broken granite surface to receive the concrete, but below that level, where the rock had not been exposed to weathering, the trimming covers a much smaller proportion of the whole surface. The underlying formation is very tight. The foundation below the former stream level is being prepared to receive the concrete, largely by sand blasting, which removes scale and soft pits satisfactorily and roughens the polished surfaces. Loose sand and dust are removed by washing and brushing with wire brushes. A more detailed description of the methods of stream control and excavation has already been published.*

Concrete Plant.—Bulk cement is delivered in box cars on a siding just above the dam site, and is unloaded into a hopper by means of a scraper drawn by a small motor-driven winch. Screw conveyors carry it from the hopper to a storage bunker which has a capacity of 15 000 bbl., and two similar conveyors extend from the bunker to the weighing house, in which are two automatic weighing machines. The machines discharge by gravity through 8-in. pipes to the two 2-yd. mixers.

TABLE 2.

Items.	Approximate quantities, in cubic yards.	Unit Prices:	
		Below original stream bed.	Above original stream bed.
Excavation in dam foundation:			
Earth and loose rock	65 000	$8.00	$1.50
Solid rock	100 000	11.20	3.00
Cyclopean masonry, 1 : 3 : 6 concrete with at least 10% of plums	300 000	11.80	11.00
Mass concrete, 1 : 2½ : 5	75 000	15.00	14.30

The rock crushers and sand pits are located in the valley. The rock and sand are hauled on a narrow-gauge railway to bunkers near the work, and lifted by belt conveyors into bunkers above the mixers. These bunkers discharge by gravity into the measuring boxes and thence to the mixers.

The elevator tower is built just up stream from the up-stream face of the dam, at the former stream-bed level. It is a timber structure, 180 ft. high, with four hoists. The concrete is distributed through steel chutes.

Contract Prices.—The contract for the construction of the Hetch Hetchy Dam was awarded on August 1st, 1919, to the Utah Construction Company,

* *Engineering News-Record*, August 11th, 1921, p. 222.

at a total estimated price of $5 447 792. The principal items are as given in Table 2.

Lake Eleanor Dam

Lake Eleanor is ultimately to be the second largest reservoir of the Hetch Hetchy System. In order to provide storage to carry the Early Intake power plant of the Lower Cherry Development over the annual low-water season, a low dam was constructed on the site of the future high dam, and will form the up-stream toe of the future structure. Characteristics of the Lake Eleanor development are given in Table 1.

The topography of the site necessitated a rather long structure, but other conditions were unusually favorable. The bed-rock was exposed throughout, sand and gravel deposits lay close at hand, and form lumber was manufactured near the work.

Design of the Dam.—Speed of construction, with minimum transport of material, was essential to impound sufficient water to operate the power plant during the dry season of 1918. These considerations led to the choice of a dam of the buttressed arch type.

The completed structure is 1 260 ft. long, the maximum height being 70 ft. above the stream bed. The maximum depth of water at the up-stream face is 60 ft. The dam contains 11 640 cu. yd. of concrete and 262 000 lb. of reinforcing steel. There are twenty arches of 40-ft. span and 460 ft. of gravity wall. Of this wall, 200 ft. is constructed as a spillway, with removable flash-boards. A log chute is also provided in this section. Over the dam is a reinforced concrete slab roadway, 12 ft. wide, with galvanized-iron railings.

The stored water is withdrawn through two 24-in. sluice valves, placed on the face of the dam. In addition, two 24-in. scouring valves are placed at the bottom of the dam.

The face of the arch is inclined at an angle of 50° from the horizontal, except the upper 7 ft., which is vertical. The radius of the arch barrel is 23 ft. in the horizontal section. The thickness normal to the face is about 3 ft. at the bottom and 15 in. at the top. An unusual feature of the design is that the horizontal section of the arch, and not the normal section, is a circular arc.

The dam was designed under the writer's direction by R. P. McIntosh, Hydraulic Engineer, assisted by R. J. Wood, Structural Engineer. Lars R. Jorgensen, M. Am. Soc. C. E., Consulting Engineer, and L. H. Nishkian, Assoc. M. Am. Soc. C. E., Structural Engineer, also collaborated. The analysis of the principles of the design has been given in a paper by Mr. McIntosh.*

Construction of the Lake Eleanor Dam.—Construction was carried out by day labor. Excavation in the foundation was commenced on September 1st, 1917, and continued to completion with the use of hand derricks. The foundations for the arches and buttresses vary from 5 to 15 ft. in depth below the surface.

Form work followed the excavation closely and concrete pouring in the footings was started on November 4th, 1917. The endeavor was to build the

* *Engineering News-Record*, September 4th, 1919, p. 464.

FIG. 7.—LAKE ELEANOR DAM, NEARING COMPLETION.

dam to a height, before a heavy snowfall, sufficient to enable construction to proceed in the following spring without interference from the anticipated flood waters. Concrete pouring was continued until December 22d, 1917, and six temporary 4 by 2½-ft. openings were left in the face of the dam through which floods could pass. From December 22d, 1917, until April 22d, 1918, all work was shut down.

At the sand and gravel pits, traps were built of sufficient height to permit of the installation of screens and the loading of trucks by gravity. The sand and gravel were delivered to large bunkers, slightly higher than the crest of the dam. Under the bunkers, narrow-gauge side-dump cars were operated by gravity to a 1-yd. mixer on the crest of the dam. The mixer discharged into 1-yd. bottom-dump cars which were handled to and from the concrete-placing chutes by endless cables, operated by single-drum steam hoisting engines.

The arch forms were carried up in sections about 10 ft. in height. Trestle work to support the forms was raised in bents 20 ft. in height, as the work progressed, to a point 8 ft. above the floor-slab.

The up-stream face of the dam was finished with a ¾-in. coat of gunite. The dam was completed late in 1918, but had already been put to use for storage of water to operate the Lower Cherry Power Plant. The entire cost of the structure was about $320 000, including the cost of a 12-mile wagon road from Hetch Hetchy, which was $28 000.

The reservoir capacity, which is about 25 300 acre-ft., is only a small part of the annual run-off from the Lake Eleanor water-shed, and the reservoir is filled each year early in the flood season.

The Aqueduct Tunnel

Construction work is well advanced on the 18.3 miles of main aqueduct in tunnel between Early Intake and the regulating reservoir at Priest. This tunnel is to carry 400 000 000 gal. per day, or 619 cu. ft. per sec., the ultimate development of the Hetch Hetchy Project.

The easterly 6 miles is through very hard, sound granite and granodiorite, and is to be left unlined except at a few points where seams have been encountered.

The greater part of the remaining 12 miles is in a slate formation, with its strike generally at an angle of less than 45° from the tunnel line, and with very steep angles of dip. The rock, in general, stands well, only occasional short stretches requiring timbering, but it is blocky and seamy, giving a large overbreak. Concrete lining of a minimum thickness of 6 in. will be used throughout this section. Where the external pressure due to heavy ground or water requires it, the lining must be strengthened to resist such pressure, by increasing the thickness of concrete, or by the use of steel reinforcement, or both. The standard lined section shown in Fig. 8 has the following properties:

Area inside of prescribed line, in square feet.....	105.63
" " " lining (*A*), in square feet.....	87.94
" of concrete inside of prescribed line, in square feet..	17.69

Excavation inside of prescribed line, in cubic yards per linear foot	3.912
Concrete inside of prescribed line, in cubic yards per linear foot	0.655

Hydraulic Elements:

Wetted perimeter (p), in feet	33.79
Hydraulic radius $\left(r = \frac{A}{p}\right)$, in feet	2.60
" slope (s)	0.00121
Coefficient of roughness in Kutter's formula (n)	0.014
C, in Chezy formula	125.5
Velocity of water ($v = C\sqrt{r\,s}$), in feet per second	7.04
Quantity of water flowing ($Q = Av$), in cubic feet per second.	619
C, in Hazen-Williams formula to give same values of v and Q.	110

The standard unlined section, shown on Fig. 9, has the following properties:

Cross-sectional area inside of prescribed line, in square feet.	167.8
Excavation inside of prescribed line, in cubic yards per linear foot	6.21

Hydraulic Elements:

Wetted perimeter (p), in feet	48.0
Hydraulic radius $\left(r = \frac{A}{p}\right)$, in feet	3.50
" slope (s)	0.00118
Coefficient of roughness in Kutter's formula (n)	0.032
C, in Chezy formula	57.4
Velocity of water ($v = C\sqrt{r\,s}$), in feet per second	3.69
Quantity of water flowing ($Q = Av$), in cubic feet per second.	619
C, in Hazen-Williams formula to give same values of v and Q.	48.5

Description of the Tunnel Line.—The tunnel is divided into eleven sections by a river canyon, seven adits, and two shafts. (Fig. 1.) For convenience, each section has been given a number as a separate tunnel. Each adit is named by the numbers of the two tunnels to which it connects. The distances between points of access are as given in Table 3.

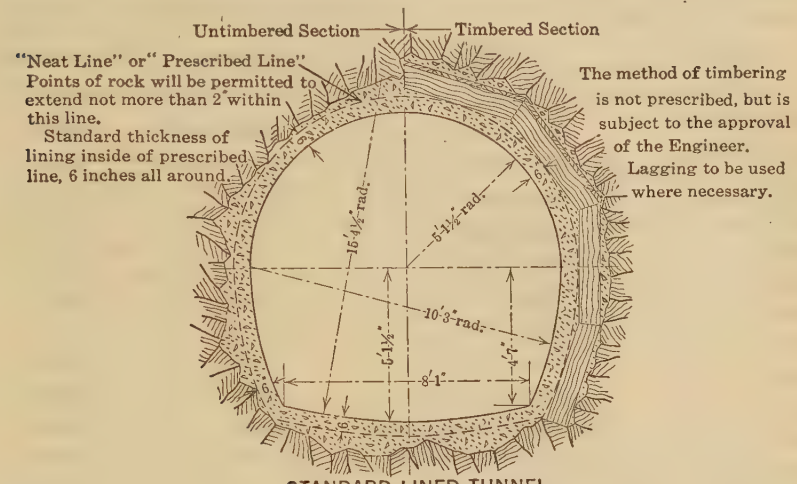
Tunnel No. 1 penetrates a ridge the summit of which is 1 750 ft. above the tunnel grade, between the main Tuolumne River and its South Fork. The latter stream will be spanned by a reinforced concrete aqueduct structure. Tunnel No. 1 is being excavated from both portals.

The line thence follows along the south wall of the river canyon to Adit 8-9, and is in from the surface only far enough to assure sound rock and sufficient cover at all points to resist the internal hydrostatic pressure. The work in this distance is being carried on from the west South Fork portal and Adits 5-6 and 8-9. The other five adits will be used only for ventilation and dumping.

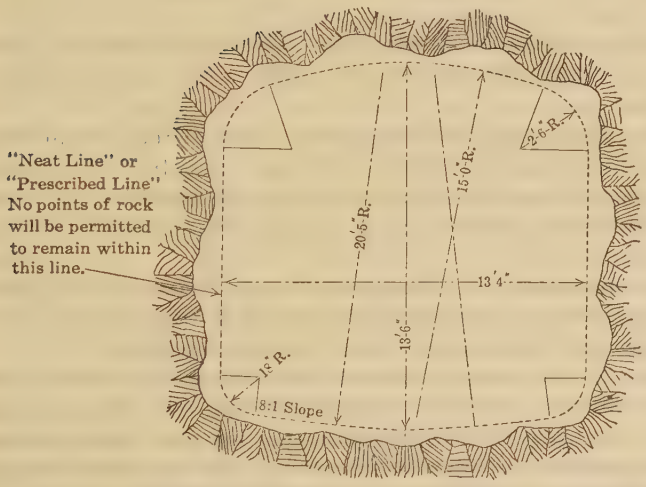
At Adit 8-9, the river turns sharply to the north. The aqueduct, leaving the canyon, continues 9 miles westerly, at an average depth of about 1 000 ft. below the surface, to Priest Portal. This section is broken by two shafts, and will be lined throughout with concrete. Excavation is progressing in

both directions at Shaft No. 1, and easterly from Priest Portal. Shaft No. 2 is only 65% completed.

Tunnel lining will not be commenced in any heading until the completion of the excavation in the heading, in order to avoid interference of operations. The concrete is to be placed by a pneumatic method.


Untimbered Section
Timbered Section
"Neat Line" or "Prescribed Line". Points of rock will be permitted to extend not more than 2" within this line.
Standard thickness of lining inside of prescribed line, 6 inches all around.
The method of timbering is not prescribed, but is subject to the approval of the Engineer.
Lagging to be used where necessary.
16'-4½" rad.
5'-1½" rad.
10'-3" rad.
6'-1½"
4'-7"
8'-1"
6"
6"

STANDARD LINED TUNNEL
MAIN AQUEDUCT—MOUNTAIN DIVISION
FIG. 8.



MAIN AQUEDUCT—MOUNTAIN DIVISION
STANDARD UNLINED TUNNEL
FIG. 9.

Cost-Plus-Fee Contract.—In August, 1917, bids were received for constructing the aqueduct tunnel from Early Intake to Priest Reservoir, but were rejected as excessive. As it was improbable that better bids could be secured by re-advertising during the period of the World War, under the

uncertain conditions then existing as to labor and material costs and financing, the work was taken up by day labor. This system proved economical, and would have been continued but for the ever-increasing difficulties of financing.

TABLE 3.—Lengths of Sections of Hetch Hetchy Aqueduct Tunnel.

Point of access.	Designation of section.	Length, in feet.	Length, in miles.
Early Intake Portal.			
	Tunnel No. 1.....	23 698	4.488
South Fork Portal.			
	Tunnel No. 2.....	3 010	0.570
Adit 2-3.			
	Tunnel No. 3.....	1 783	0.338
Adit 3-4.			
	Tunnel No. 4.....	2 164	0.410
Adit 4-5.			
	Tunnel No. 5.....	2 785	0.527
Adit 5-6.			
	Tunnel No. 6.....	4 362	0.826
Adit 6-7.			
	Tunnel No. 7.....	5 676	1.075
Adit 7-8.			
	Tunnel No. 8.....	4 964	0.940
Adit 8-9.			
	Tunnel No. 9.....	12 472	2.362
Shaft 1.			
	Tunnel No. 10.....	15 320	2.902
Shaft 2.			
	Tunnel No. 11.....	20 489	3.880
Priest Portal.			
.....	.....	96 723	18.318

Bids were again received in April, 1920. The cost records and other information gained during the previous two years were made available to prospective bidders.

Although it has always been, and still is, the policy of the City to award contracts for construction on the definite unit price basis, the advantages of the cost-plus-fee form of contract, under the abnormal conditions then prevailing, led to alternative bids being sought under both the unit price and cost-plus-fee plans.

The total of the guaranteed maximum costs of work under the cost-plus-fee form of contract proved to be nearly 20% less than on the flat-unit-price basis. Accordingly, the contract was awarded on the cost-plus-fee basis.

In this contract, the contractor guarantees that the costs of the respective items of the work will not exceed the unit prices fixed in his bid as maximum unit costs. He is not made fully responsible for excess over these maximum costs, but is only penalized to the extent of a part of his fee, which, however, amounts to a sufficient sum to give him a lively interest in holding down costs.

The total fee is a fixed sum. A part is payable at the beginning of each year of the life of the contract, to cover the expense of financing the year's work. The remainder is payable in quarterly installments. Of each quarterly installment, 25% is retained by the City until the termination of the contract, and the accumulation of these retentions is the maximum sum that the contractor is to forfeit if the costs exceed the guaranteed maxima.

Detailed cost records are kept by the City's accounting force, and the contractor has the right to examine these records at any time. The determination of costs under the contract items, for comparison with the guaranties, is to be made annually. General costs not directly chargeable to particular items, and the fee itself, will be apportioned *pro rata* to the items. The legality of this form of contract was questioned in a taxpayer's suit, and was affirmed by the Supreme Court of the State of California.

The total of the guaranteed maximum prices under the contract, as awarded, was $7 802 952. The guaranteed maximum costs per foot average about $67.50 for unlined and $82.50 for lined tunnel.

Tunnel Excavation Methods and Equipment.—In all the tunnels, the "round" covers the entire face. The spacing and pointing of the holes varies somewhat in the different headings, partly because of the differences in formation, and partly because better results are obtained by allowing some latitude to the foreman at each tunnel camp. Fig. 10 shows the drilling diagram for the two South Fork headings which are in hard, fine-grained granodiorite. Here, forty-two holes are used, and are shot in six stages by means of delay-action electric exploders.

After shooting, the muck is cleared from the upper half of the face, holes are drilled in the side-walls, and a staging is constructed, supported on bars inserted in the side-holes. Drilling in the upper half then proceeds simultaneously with the mucking. Mucking machines are used in all headings, and 4 and 5-ton electric storage battery locomotives are used to haul the muck trains.

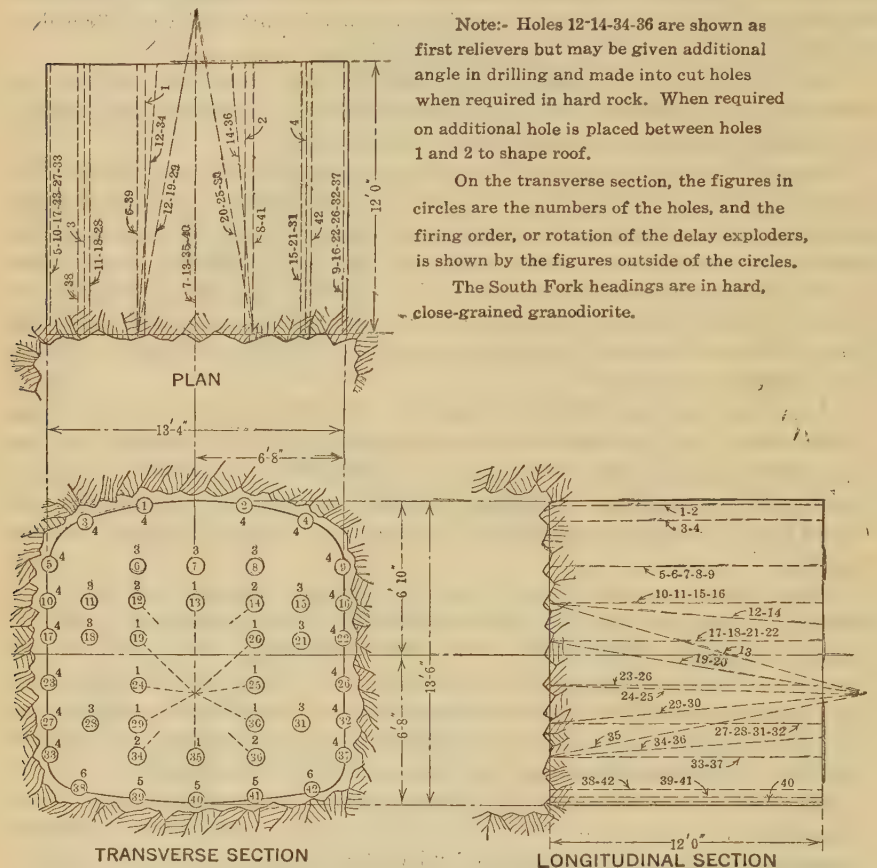
The rate of progress of excavation is from 300 to more than 700 ft. per month, varying according to the formation. The record for one month, 776 ft., was made in August, 1921, in the Priest Heading (west end of Tunnel No. 11). This heading is in altered slate and schist, and is being driven according to the standard section shown in Fig. 8, working three shifts, and drilling 25 holes per round. The excavation to August 31st, 1921, totaled 37 273 ft. for the ten headings in which work is now in progress.

The men are working under a bonus system, under which they receive extra pay in proportion to the excess of the average progress over a predetermined standard rate of progress per shift. The bonus, however, is subject to deduction in case the pay-roll, powder expense, and (in tunnels which are to be lined) the estimated cost of concrete required to fill overbreak, exceed certain fixed amounts per foot of tunnel.

Shafts.—The two shafts are alike in design. Each has three compartments: Two hoisting compartments, each 4½ by 4½ ft. inside of timbers, and a manway, 4 by 4½ ft., in which the ladders and pipes are placed. The manway has been found to be too small for convenience. Self-dumping skips with capacities of 32 cu. ft. are used.

Arrangements at Bottom of Shaft No. 1.—At Big Creek, the tunnel level is 575 ft. below the collar of the shaft. The shaft extends 71 ft. farther down, making the total depth 646 ft. The muck from the headings is dumped into a rock pocket, about 30 ft. deep, with a capacity of 3 000 cu. ft. At the bottom of the rock pocket, there are two pneumatic cylinder-operated gates through

which the muck can be discharged into either hoisting compartment of the shaft. These gates are operated by an attendant at the level of the skip landing, and the usual time of loading a skip is 15 sec. At the same level is the pumping station, for which the lowest 30 ft. of shaft is the sump.



MAIN AQUEDUCT - MOUNTAIN DIVISION
DRILLING DIAGRAM : SOUTH FORK HEADINGS

FIG. 10.

The neat tunnel section being driven from this shaft is 11 ft. 3 in. wide, outside the lining. At the shaft, it is widened out to 16 ft. for a length of 120 ft., and two or three recesses are also provided, giving space for charging the storage battery locomotives and for storing and repairing locomotives, cars, mucking machines, or other equipment which is too bulky to be hoisted conveniently to the surface.

SUMMARY OF WORK COMPLETED

The principal achievements on the project thus far include: Purchase of reservoir lands, water rights, rights of way for the Hetch Hetchy Railroad

and for the main aqueduct in the Sierra Nevada, and lands for the Groveland Headquarters and Moccasin Creek Power Plant; construction of the Hetch Hetchy Railroad and purchase of rolling stock; construction of the Lake Eleanor Dam; construction of the Lower Cherry Power System, including a 3½-mile aqueduct, 3 000-kva. power plant, and 33½ miles of transmission lines; 8 miles of main aqueduct tunnels have been excavated; at the Hetch Hetchy Dam site, excavation to foundations has been completed, and the pouring of concrete has been begun; and the outlet valves for the Hetch Hetchy Dam have been constructed.

The expenditures to the present time total about $14 000 000. The number of employees on the construction work of the entire project averaged about 1 377, during the fall of 1921.

The Engineering Staff of the Hetch Hetchy Development includes the writer who is Chief Engineer, and N. A. Eckart, M. Am. Soc. C. E., Engineer, Hetch Hetchy Project. The City Office staff includes Leslie W. Stocker, M. Am. Soc. C. E., Assistant Engineer; R. P. McIntosh, Hydraulic Engineer; R. J. Wood, Structural Engineer; Paul J. Ost, Electrical Engineer; and, Edwards P. Jones, Mechanical Engineer. On the Construction Staff are: C. R. Rankin, M. Am. Soc. C. E., Construction Engineer, Hetch Hetchy Dam; and, Lloyd T. McAfee, Construction Engineer of the Aqueduct tunnels, and Superintendent of the Hetch Hetchy Railroad.

DISCUSSION

JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).—Tunnels for inspection purposes and drainage, such as those described by Mr. O'Shaughnessy, have been used in high masonry dams for many years. They were used in the Vyrnwy Dam and, perhaps, in earlier dams. They have become standard practice; but the use of drainage wells, lined with porous concrete blocks, such as those in the Hetch Hetchy Dam, is quite recent.

The writer was connected with the construction of the Olive Bridge Dam of the Catskill Water Supply for New York City, in which, so far as he has been able to determine, the drainage well with porous concrete block lining was first used. These wells in the Olive Bridge Dam were about 12½ ft., center to center, throughout the entire length of the masonry section, and were located a short distance back from the up-stream face. The wells were formed by using porous pre-cast concrete blocks, with a hole cored through the block.

The theory of the action of these drains is that the water which forces its way through the interstices of the concrete of that part of the dam up stream from the drains, will be conducted through the porous block to the well in its center. Thus, most of the upward pressure which would otherwise exist on every horizontal plane through the dam, is assumed to be relieved.

The drainage wells are useful and, undoubtedly, accomplish a great deal; but, in the writer's opinion, the porous block is a delusion and a nuisance. At the Olive Bridge Dam, at first, these blocks were moulded of 1:4:14 concrete and the mixture was kept as dry as possible, in order to insure porosity. Even at an age of several months, so many of these blocks were broken that it was necessary to increase the strength of the mixture so that they could be handled with some degree of safety by the derricks. The placing of the blocks was always troublesome, because they were so fragile. When they were firm enough to handle, it was generally because the concrete was more dense than it was supposed to be, and the blocks were not porous.

When the porous drainage blocks were set, it was endeavored to heap around them the dryest concrete available, but, in spite of all precautions, the mortar from the Cyclopean masonry being placed on the dam would run into the pores of the block and clog them. Because of the convenience of the drains as catch-alls, it was impracticable to prevent the workmen from throwing into them sticks, laitance, and surplus grout. This effectively lined many of the drains with an impervious coating and, in some cases, they were entirely clogged and to open them, a length of 90-lb. rail suspended from a derrick was worked up and down like a drop-hammer pile-driver. It is the writer's belief that, in spite of the care exercised to secure porous blocks, there are few of them in the Olive Bridge Dam that are any more porous than the concrete surrounding them.

If this is so, why use porous blocks to form the drainage wells? When effectively porous, they have little strength; therefore, why not discard them and have drainage wells, say, 30 in. square, which is about the size of a

* Chf. Engr., Depts. of Hydro-Elec. Power Plants, Water Supply, and Sewerage, The Ludlow Engrs., Inc., Winston-Salem, N. C.

porous block, located about 12 ft., center to center, along the dam? With the increase in the width of the wells, their effectiveness as vertical drains would be greatly improved. The drains could be built by using forms, and after the dam was finished the relatively impervious surface next to the form could be broken up by bush-hammering, as the size of the well would permit a small man to work in it with an air-hammer.

The writer is pleased to note that bulk cement is being used on the Hetch Hetchy Dam. In 1915-16, he was connected with the construction of the Wissota Hydro-Electric Project in Wisconsin, where bulk cement was used, and since then he has been convinced that for any job where a large quantity of concrete is to be placed from a single plant, there is a great saving in the use of bulk cement. In 1915, its use was a novelty, but the indications are that it will soon become standard practice for such work.

Allen Hazen,* M. Am. Soc. C. E. (by letter).—The Hetch Hetchy water supply for San Francisco is an important undertaking. The difficulties of the aqueduct line will be evident to water-works engineers from an inspection of Plate VIII, which shows a 31-mile tunnel through the Diablo Range, and many miles of large-size high-pressure pipe line.

The writer in speaking before the Chamber of Commerce of Oakland, Calif., once stated that in order to secure cheap water in the San Francisco District from a mountain source comparable with that obtained by other Western cities, it would be necessary to arrange with the geologists to turn the Diablo Range (represented in Plate IX, in the Coast Range Division) through an angle of 90°, so that the line of the aqueduct would be parallel, instead of at right angles, to the mountains.

The difficulties of supplying San Francisco and the adjoining communities with water are perhaps greater than those involved in the supply of any other American city. Many of the larger cities have been built near abundant water supplies. Sometimes, the water has been the reason for the location and growth of the city; but San Francisco was built on a harbor, which is one of the finest in the world, and its excellence is sufficient reason for the growth of the city. There are incidental disadvantages in this particular location, and one of them is that a water supply must be secured by overcoming unusual obstacles.

At present, the San Francisco District is supplied from sources much nearer the city than the Hetch Hetchy Reservoir, and these cheaper sources no doubt, will continue to furnish the supply for many years. The ten years suggested by the author may pass, and several other like periods, before it is necessary to build the Hetch Hetchy Aqueduct. The climate of San Francisco does not lead to the large per capita use of water required in some other Western communities. The present per capita use is low, and the climate and the habits of the people are such that it is likely to remain low.

The local sources of water supply have considerable reserve capacity. With reasonable development, the supplies now owned by the Spring Valley Water Company, and held in reserve by that Company, will cover three times the

* Cons. Engr. (Hazen, Whipple and Fuller), New York City.

present rate of output. This includes the Calaveras Reservoir, which alone will produce more than the entire present supply, and two other large prospective reservoirs on tributaries of Alameda Creek. Even the coast streams from the western slope of the peninsula south of San Francisco, although discarded in several engineering reports, are capable of furnishing a considerable quantity of water through the present Spring Valley System, at a unit cost that would be only a fraction of that of Hetch Hetchy water.

The reserves for the cities across the Bay, served by the East Bay Water Company, including Oakland and Berkeley, are not as great, but San Francisco is not building the Hetch Hetchy for them.

San Francisco may need the Hetch Hetchy badly at some future time. There is a wide belt of dry country beyond the present local sources and between them and the Sierras. If San Francisco did not now claim and hold a Sierra source, the time might come when she would be in need of water. Nothing can be said against the far-sighted policy of reserving an adequate supply of mountain water, but having said this, it is well to keep in mind also that when it is necessary to go for it, it will be very expensive water. It is for the best interests of the city that all the good cheap water near at hand should be developed before so much money is spent on a distant and costly source.

The author mentions briefly the proposed arrangement by which water from Calaveras is to be allowed to flow down through natural channels to Sunol, and through the present Spring Valley Aqueduct to Niles and then taken by pipes to a section of the Hetch Hetchy Aqueduct built in advance of the remainder, to carry the water to the present Crystal Springs Reservoir. The writer understands that by this procedure all, or nearly all, of the 700 ft. of elevation at which Calaveras water is available, is to be lost. The water is to be taken practically to sea level and is then to be pumped into the Crystal Springs Reservoir at Elevation 283. Most of it will have to be pumped again before it reaches San Francisco.

It may be interesting to note in this connection that at San Francisco the mountains are near the sea, and elevations are high. The main distribution reservoir in the present system is at Elevation 365, and one-half the total output is used at this elevation and above it. The cost of pumping from sea level to these great elevations is a matter of no small importance.

The best use of available resources would be to carry the Calaveras water through an aqueduct to be built on solid ground, south of San Francisco Bay, extending to a point on the hills back of Palo Alto, from which point a grade aqueduct could be built to the San Andreas Reservoir at Elevation 440, and thence by tunnels and pipes to the city at Elevation 365. Such an aqueduct would be a few miles longer than the route now proposed by the author, but, as it would avoid a difficult submarine crossing, there would be little, if any, additional first cost. By its use, Calaveras water, in a quantity more than the entire present supply, could be delivered at the Honda Reservoir level of 365 ft., without pumping. The water of the Crystal Springs Reservoir would flow by gravity to a lower distributing reservoir, and the only pumping needed to operate the entire system would be for ground-water from the Alameda

County sources and for water required in high-service districts above Elevation 365.

The reason for the selection of the route proposed, as stated by the author, is that it will "make available, without duplication of aqueducts, the additional Spring Valley water". This brief explanation hardly does justice to the situation. All the future growth in San Francisco that can be reasonably anticipated in financial arrangements now made, will not warrant a larger aqueduct than can be filled with Calaveras water. The business does not warrant a larger one, and neither the water company nor the city can afford to build now for greater capacity.

Calaveras water will fill such an aqueduct for all time. If such an aqueduct is used at any future time to carry Hetch Hetchy water, it can only be accomplished by wasting Calaveras water. There is no reason why Calaveras water should be wasted to make way for other water. The supposed saving by using a part of the Hetch Hetchy Aqueduct for Calaveras water, therefore, has no real basis.

The procedure of taking Calaveras water through part of the Hetch Hetchy Aqueduct with a sacrifice of its head will result in an increase in pumping costs and in investments in pumping stations, that will amount to great sums of money. Water may be supplied in this way; but water rates will have to be adjusted accordingly, and the consumers will pay for the economic waste.

Fred A. Noetzli,* Assoc. M. Am. Soc. C. E. (by letter).—The history of the Hetch Hetchy Project is rich in incidents which interfered more or less with the progress of the work. Opposition for political reasons and hostility against municipal ownership of public utilities, united themselves for fighting this project, and, in addition, at certain times criticism was heard from various quarters with regard to purely engineering questions. Those times of uncertainties have passed, however, and the first unit of this great project is in full course of construction, uniting friends and former opponents in the common desire for a successful completion of the work.

The main single structure of this project is the Hetch Hetchy Dam, a structure of the curved gravity type, which will ultimately be more than 300 ft. high. For the initial dam, it is proposed to discharge the flood waters through a siphon spillway near the center of the crest, where the dam is highest. The writer would like to know the reasons for the adoption of a siphon spillway in preference to a spillway with radial gates, which was proposed as an alternative scheme when bids were asked for the construction of the dam. As the dam is to be raised later, all the siphons will be covered, and practically lost, whereas gates might have been salvaged and used again for the spillway of the raised dam.

It will be interesting to observe, during periods of floods, the effect of water discharging from the siphons and over the stepped-off down-stream face of the dam. The water will issue from the siphons at a velocity of about 25 to 30 ft. per sec. Assuming a maximum flood flow of 12 000 sec-ft., the total kinetic energy of this water, in dropping from the reservoir surface at

* Chf. Engr., Beckman and Linden Eng. Corporation, San Francisco, Calif.

Elevation 3 720 to the stream bed at Elevation 3 500, would be between 200 000 and 300 000 h. p., and that part of it which is expended on the dam no doubt will produce some vibrations.

This will afford an excellent opportunity for having such vibrations measured by recording instruments. At the same time, such tests may permit conclusions to be drawn as to the effect of siphoning over the crest of slender arch dams and the vibrations incidental thereto. It is believed that the large body of water back of the dam will reduce, if not practically nullify, such vibrations, because the combined mass of the dam and the water for a certain distance from the dam is so enormous as to make insignificant the stresses from vibrations caused by spilling. However, this assertion can probably be proved or disproved only by actual measurements, and it is to be hoped that such tests will be made on the Hetch Hetchy Dam in the interest of future structures to be built.

At the time when the Hetch Hetchy Dam is to be raised, the peculiar problem will be encountered of preventing any seepage water from accumulating in the planes of contact between the old and the new concrete, in order to prevent uplift for the added parts of the dam. In the writer's opinion, particular difficulties will be encountered in sealing tightly the smooth channels of the siphons, mainly on account of the natural shrinkage of the newly poured concrete which will tend to loosen it from the cover of the spillway channel and thus admit water to the stepped-off parts of the old down-stream face of the dam.

The Lake Eleanor multiple arch dam is an excellent representative of this type of structure. The dam is at an elevation of 4 600 ft., in a region of ice and snow. It would be of interest to know the effect of freezing and thawing, if any, on the slender arches of this dam, also the effect of ice pressure. It is to be hoped that the City of San Francisco will make funds available for measuring temperature, deflections, and stresses in the dams of the Hetch Hetchy water supply project, in order to furnish data which may be used to advantage when the time comes to raise these structures.

C. R. Rankin,* M. Am. Soc. C. E. (by letter).—After conducting certain experiments with porous concrete blocks, such as are used for drainage wells in the Hetch Hetchy Dam, the writer is convinced they will perform their function of providing a free outlet for any seepage water which may enter the structure from the up-stream face.

Drainage wells, consisting of porous blocks, 3 ft. 3 in. square, outside measurement, and 3 ft. high, with a 15-in. vertical opening, are placed approximately 12 ft. on centers above the stream bed. Also, a row of drainage wells below the stream bed, in the nature of sub-drains, is carried from a continuous sub-drain constructed of the same class of porous blocks placed in the bottom of the cut-off trench, and is given outlet in the inspection gallery at stream-bed level. These wells are placed about 50 ft. on centers.

The main system of drainage wells is started from the lower inspection gallery at stream-bed level, and approximately 33 ft. from the up-stream face,

* Constr. Engr., Hetch Hetchy Water Supply, Hetch Hetchy, Calif.

and is carried up on a batter of 1 on 25 toward the up-stream face. These wells connect into the upper inspection gallery 185 ft. above. From the upper inspection gallery, an upper row of drainage wells is placed, with the same spacing, leading to the top of the structure.

For the dam 1 600 porous blocks were required, and these were made with steel collapsible forms in a block plant ½ mile from the dam. A mixture of 1 part cement, 1 part sand, and 8 parts of crushed rock, ranging from ¼ in. to 1 in. in diameter, made a strong and very porous block. The forms were removed in 3 days, and after curing for 7 days, the blocks were stacked in rows, five blocks high, by derrick. To facilitate handling by derrick to the storage piles, and, later, to cars and thence to a position on the dam, two grab holes are placed 6 in. from the top of block and 1 in. deep. In making the 1 600 blocks and placing the 750 blocks, not one has been broken, although some have been dropped several feet.

The practice in placing the blocks is to carry them from two to three blocks above the concrete level, and to cover the 15-in. opening with a lid of 1-in. lumber, to exclude débris. No débris has been thrown in the wells, as it would fall directly to the inspection gallery and be in evidence.

No difficulty has been experienced in excluding grout from the pores of the blocks, as has been demonstrated by the experiment of pouring the concrete over the block, permitting it to take its initial set, and then removing the block and chipping off the surface. Penetration of grout into the block did not exceed 1 in. in depth. Examination of the interior of the placed blocks by entering the 15-in. opening, shows no tendency of the pores to fill with grout. Tests as to the porosity of the blocks were conducted at the time they were made, by turning the flow from a 2-in. pipe on the side of a block. The full flow passed through to the core opening. During the wetting down of the concrete on the dam, a large quantity of the water on the surface flows freely through the porous blocks.

The great advantage of placing porous blocks to form a drainage well is the establishment of a large drainage perimeter without creating a plane of weakness in the structure and without sacrificing weight. Also, the large, effective drainage perimeter is established at little extra expense over the cost of filling the same volume with dense concrete. The weight of the porous concrete blocks is about 3 440 lb. per cu. yd., and their compression strength on 28-day test is 994 lb. per sq. in.

With the system in use on the Hetch Hetchy Dam for placing porous blocks, no difficulty has been experienced in securing a satisfactory result. As a means of securing a tight joint between the new and the old concrete, at the time the dam is raised, the writer believes that grout pipes will be most satisfactory. These grout pipes would not be charged until sufficient time had elapsed to insure that full shrinkage had taken place in the newly placed concrete.

M. M. O'Shaughnessy,* M. Am. Soc. C. E.—Attention has been directed by Mr. Hazen to the physical difficulties which confront San Francisco in securing a cheap water supply for domestic purposes.

* City Engr., San Francisco, Calif.

He is particularly well qualified to pass on this subject as he was employed by the City to make a report in 1911, on a filtered river supply from the San Joaquin or Sacramento Rivers as a substitute for the proposed Tuolumne water supply.

In his report on December 4th, 1911, he recommended intakes at Rio Vista, on the Sacramento River, about 10 miles north of Antioch, for a daily supply of 60 000 000 gal. As irrigation and water supply conditions then existed, this recommendation was wise. Conditions in the matter of water supply, however, are rapidly changing in California. A recent discussion before the Society on the salt content of the waters of the Lower Sacramento and San Joaquin Rivers explains the changes in the past eleven years.* In that period more than 40% of the 1 500 000 acres in the Sacramento Valley has been placed under irrigation and a large quantity of water has been diverted for this purpose. This has resulted in litigation by the lower land owners and the Town of Antioch, to restrain the upper diversions of water, so as to preserve the quality of the water free from salt in the vicinity of the lower communities. In the face of those conditions, therefore, San Francisco would have been unwise to adopt a supply of this kind as a permanent source of water for a growing community.

The condemnation 10 and 20 years ago of the Lower Sacramento and San Joaquin Rivers by John R. Freeman, President, Am. Soc. C. E., and C. E. Grunsky, M. Am. Soc. C. E., as a permanent source of water supply for San Francisco is well founded. There can be no argument against the conclusion that a near-by source of water supply, 60 miles distant from a community, will be cheaper than a source 160 miles distant. The public and taxpayers of San Francisco were thoroughly enlightened as to this fact during the discussion of alternative water supply systems 12 years ago, when Hetch Hetchy was approved by them practically unanimously. They knew the cost would be higher, but they were willing to pay the price for a superior quality of water.

In Mr. Hazen's report of 1911, he gives the average hardness, in parts per million of calcium carbonate, as follows:

Present supply of San Francisco	100
San Joaquin River at Lathrop, near Stockton	90
Sacramento River at Rio Vista	55
Lower Tuolumne River at La Grange, including the more highly mineralized water from the foot-hills	40
Hetch Hetchy Reservoir.....not more than	20

Quoting Whipple on the value of water,† Mr. Hazen calculated an allowance of 10 cents for each part per million, as an approximate estimate of the cost of hardness in a public water supply. To supply 60 000 000 gal. daily, this totals $480 per day, or $175 000 per annum, which amounts to 5% on an investment of $3 500 000. For the supply of 400 000 000 gal. daily, which the Hetch Hetchy project will ultimately develop, this would make an equivalent in its favor, on water hardness alone, of $22 750 000.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 455.

† "Whipple's 'Water Supply'", p. 27.

Power Output

Although hydro-electric power is a by-product of the water-supply project, it has been brought to an earlier development than was originally contemplated in the Freeman report, in order that the revenue derived from the sale of power might be applied to the payment of interest charges on the outstanding bonds.

The increase in the price of oil from 60 cents per bbl., 10 years ago, to $1.40 per bbl., at present, has created enhanced value for the sale of hydro-electric power. Modifying the construction program from that proposed in 1912, instead of deferring power development to the future, as originally contemplated, this commodity to the extent of 70 000 continuous h. p. will be made available within 2 years. If a judicious method is adopted for the disposal of this power the revenue received from its sale ($1 500 000 to $1 750 000 per year) will go far toward paying the expenses of the entire mountain development, and will be an immediate relief to the taxpayers of San Francisco.

Assuming power revenue for the first large unit as $1 500 000 per year, and using Mr. Hazen's basis of 5% for the value of money, this method would develop a power value of $30 000 000 for the first unit of San Francisco's construction.

Other units of power can be developed later on the Hetch Hetchy watershed at relatively lower rates per horse-power, which it is now unnecessary and futile to compute. Added, however, with the water value from softness of $22 750 000, a value of power of $30 000 000 makes a very substantial item to the credit side of the Hetch Hetchy project, which must be considered and allowed for in computing the ultimate cost of water.

The method of disposal of power has not yet been determined. Two alternatives are under consideration: First, to dispose of power wholesale from the Moccasin Creek plant to some existing public utility company with a large distributing system that will immediately absorb the output, with a guaranty of power for municipal needs at minimum prices inside the limits of San Francisco; or, second, the City may construct a transmission line costing about $3 000 000 into San Francisco, taking over immediately the load of the Municipal Railway, which amounts to $400 000 per year, and gradually acquiring a distributing system for lighting and other purposes.

The Municipal Government of San Francisco is committed to the policy of owning all the street railways, and as the franchises of the present companies expire, between 1929 and 1932, there will be an additional revenue of $1 300 000 per year from the disposal of power for the additional railway needs.

In either alternative, the first large power unit of the city will have a capital value exceeding $20 000 000, which, of course, in all fairness, should be considered as a credit in estimating the cost of the Hetch Hetchy project, compared with other alternative sources.

Mr. Hazen takes a more optimistic view about the possibilities of the sources of the Spring Valley Water Company for a future water supply than is justified after the exhaustive analysis made in 1912 by Mr. Freeman, who spent nearly two years investigating the potentialities of this and other systems. Mr. Hazen's allusion to the potentialities of Calaveras Reservoir and

two other reservoirs, Arroyo Valle and San Antonio, to supplement the immediate needs, is unwarranted. The farming communities in Livermore and Pleasanton Valleys and Niles Cone, to which the waters from those sources are tributary, are seriously alarmed over the present lowering of the water planes by the more intensive use of water for irrigation and the diversion of the Spring Valley Water Company for San Francisco's needs.

Active litigation is now being prosecuted against the Spring Valley Water Company, and although the Company is not responsible for the depletion of the underground water supply north of Alameda Creek, it is evident that the continuous pumping, at Alvarado, of 10 000 000 gal. of water daily, has caused a serious depletion of that supply.

With vision and foresight, San Francisco has engaged, therefore, in the Hetch Hetchy undertaking and is willing to pay the bill, but if one takes into consideration the assets of softer water, and immediate and future power potentialities, the water as a whole cannot be styled "very expensive water."

The population of San Francisco is now practically on water rations, 100% metered. For five months of the summer, from May to September, inclusive, San Francisco is generally without rainfall. At present, there is not an adequate and generous supply of water to keep the sewers of the city properly flushed during this period. Such a condition is intolerable for a growing American city that needs an abundance of water for manufacturing and sanitary purposes, as well as for domestic use.

Calaveras Water

A contract was entered into on March 20th, 1922, between the Spring Valley Water Company and the City of San Francisco, by which a section of the Hetch Hetchy Aqueduct, about 21 miles in length, will be immediately built between Niles and Crystal Springs. This involves a tunnel about 9 000 ft. long from a point 2 miles west of Redwood into Crystal Springs Reservoir, and the construction of 19 miles of pipe 5 ft. in diameter, with a capacity of 45 000 000 gal. daily, crossing the Bay by means of reinforced concrete pile trestle and short submarine pipe 2 000 ft. long, at Dumbarton. Of the Calaveras water, 24 000 000 gal. daily additional will be led by gravity through the Niles Conduit—to be enlarged—and by making a 3-mile extension southerly from Niles, it will meet the Hetch Hetchy Aqueduct and deliver to it under 160-ft. head, and convey it by gravity to a pumping station near Redwood City, whence the water will be boosted by pumps into Crystal Springs Reservoir through the tunnel at an elevation of 293 ft.

The estimated cost of this part of the project will be less than $5 000 000, on which the Spring Valley Water Company has agreed to pay interest at the rate of 5% from the date of commencing the work. This contract for use expires March 31st, 1933, by which time it is hoped that the City will have acquired all the distributing system and properties of the Spring Valley Water Company necessary for use in connection with the Hetch Hetchy project.

The Mayor and City Officials of San Francisco are committed to the policy of acquiring the private company. They have put the properties of the Spring Valley Water Company up three times to the public to secure permanent pur-

chase and have exhausted every personal effort to make it a success, but so far have not secured the 66% of the vote necessary under the Charter, for this purpose.

The ultimate plans contemplate a sub-aqueous tunnel at Dumbarton, through which the Hetch Hetchy gravity pressure pipes will lead directly into the Crystal Springs Reservoir and to San Francisco.

Aqueduct Route

The contention of Mr. Hazen that the gravity route which he recommends for Calaveras water, starting from an elevation of 700 ft. at Calaveras and delivering into San Francisco at the Laguna Honda Distributing Reservoir at an elevation of 370 ft., at an estimated cost of $12 000 000, was the most desirable, is a debatable problem, on which honest differences of opinion and judgment in favor of both routes can be consistently sustained.

If the Spring Valley Water Company was going to stay permanently in business, and if the City was to rely on that institution for a permanent water supply, it might possibly be a wise solution of the aqueduct route. There are one or two objectionable features to this route. Selecting the 700-ft. outlet level at Calaveras would leave 100 ft. of water at the bottom of the reservoir incapable of being drawn off except by pumping. This volume would include more than 10 000 000 000 gal. of capacity.

The San Andreas Reservoir at an elevation of 440 ft., which would be the stand-by reservoir on this gravity conduit, has a capacity of only 6 000 000 000 gal.

On the other hand, by bringing the water directly from Niles across the Dumbarton route, a line of about one-half the length is obtained, costing less than $5 000 000, making an initial capital saving of $7 000 000 and delivering water into the Crystal Springs Reservoir—of 22 000 000 000 gal. present capacity—which can be enlarged at a very moderate expense to 30 000 000 000 gal., be an integral unit of the Hetch Hetchy project, and be made a splendid stand-by source for the future greater water supply close up to the city limits.

Another advantage of the latter route is by a further extension of the Hetch Hetchy Aqueduct easterly from Niles, with 3½ miles of tunnel and 1¼ miles of pipe, to Alameda Creek, all pumping expenses can be dispensed with and Calaveras water brought by gravity to Crystal Springs and into the Amazon Reservoir at an elevation of 250 ft., inside the city limits. The still further advantage of this route is that, at certain stages of storage, Crystal Springs Reservoir will serve a dual purpose, acting as a receiving reservoir for the Hetch Hetchy Aqueduct and taking care of the fluctuations of flow in that aqueduct, with a head ample to deliver water into San Francisco.

The terminal reservoir in San Francisco of Mr. Hazen's gravity aqueduct would be the Laguna Honda Reservoir at an elevation of 370 ft. The greatest advantage of his route would be a saving of level of 125 ft. in delivering Calaveras water by gravity into San Francisco. The output from the Calaveras Reservoir, after considering riparian rights lower down, will average, say, 30 000 000 gal. daily, which, pumped against a head of 125 ft., at 5 cents per 1 000 000 gal., would represent an annual charge of $68 457.50, or 5%

interest on $1 368 000. The advantage to be gained by the higher delivery level is insignificant compared with the extra cost of building a separate conduit in the future to deliver Hetch Hetchy water into San Francisco at the 250-ft. level. Besides, more than 52.8% of the present water consumed in San Francisco is used below the 255-ft. level, and, no doubt, the extension of factories in the future along the water-front and low-lying reclaimed lands will increase this percentage. The use of water on the higher levels of the city, which reach an elevation 600 ft. above the sea, is confined exclusively to residences, with relatively low per capita consumption.

As the Crystal Springs Reservoir is more than five times the volume of San Andreas Reservoir, it will serve more effectively the future Greater San Francisco, occupying the peninsula from San Francisco to San José, with a population of 2 000 000 or 3 000 000, and its selection as an integral unit of the main delivery system is, in the writer's judgment, the wisest, and not the alternative suggested by Mr. Hazen.

Cost Comparison

Hazen Plan.—Mr. Hazen's proposed aqueduct is estimated to cost $12 000 000; the cost of the City's aqueduct, *via* Crystal Springs, is estimated to be $5 000 000 — a difference in favor of the adopted plan of $7 000 000, which, invested at 5% compounded, should be the equivalent of $15 000 000 in 15 years. The first year the interest on $7 000 000 at 5% would be $350 000.

Adopted Plan.—The City will have its own power, costing less than 1 cent per kw-hr. Pumping costs will not be more than 5 cents per 1 000 000 gal., 1 ft. high, $8.15 per 1 000 000 gal., for 163 ft. high, or, for 24 000 000 gal. daily, the approximate increased water obtained from a partial completion of the Calaveras Dam, means a cost of $195.50 per day, or a pumping charge of $71 394 yearly, which would be progressive and fluctuating with the increased demand for water.

This pumping can be dispensed with in the next Hetch Hetchy unit of construction from Irvington easterly, with the following outlay:

7 500 ft. of pipe, 5 ft. in diameter	$ 200 000
18 100 ft. of 10 ft. 3-in. tunnel	1 400 000
Branch to Calaveras Creek	250 000
Total	$1 850 000

Interest on this outlay would be less than $100 000 yearly. Its construction would eliminate proposed pumping costs, and if the urgency of completion of the present plan did not demand all attention, its construction would not be deferred.

From a financial standpoint, therefore, there is no question as to the wisdom of the City's selection.

Mr. Hazen has been consistently in favor of an accessible conduit that would not cross the Bay and, from prudential reasons, there is much to sustain his attitude in favor of this alternative route. The Spring Valley Water Company, however, has had four lines of pipe across the Bay at Dumbarton

for many years. Those pipes survived the great earthquake shock in 1906, are nearly 6 000 ft. in length, and lie in the mud overlaid by a maximum depth of 50 ft. of water. With the revised plans of the city's crossing at Dumbarton, submarine pipes will be limited to a length not exceeding 2 000 ft., or one-third the length of the Spring Valley pipes. The approaches to the submerged siphon will be built on a reinforced concrete trestle, thoroughly braced and anchored, and even greater care will be used in their construction than in that of the Spring Valley mains, so that the chances of disaster to a short submarine pipe are not regarded as very serious.

If the city was dependent on these pipes alone, Mr. Hazen's attitude would be consistent, but, fortunately, the city is fortified by the three large peninsular storage reservoirs of the Spring Valley Water Company, which, with the enlargement of Crystal Springs, will hold 40 000 000 000 gal. of water, and be an ample stand-by source to sustain a community on the San Francisco Peninsula for perhaps 100 years to come.

Siphon Spillway in Hetch Hetchy Dam

The siphon type of spillway was adopted for the following reasons:

1.—Because of the automatic feature and absolute dependability of operation, as long as the entrance openings do not become clogged. The siphons will be in action only during the season of melting snows (April to June), so that no trouble will be experienced from ice conditions, and as the reservoir site has been cleared of trees and brush, the possibility of clogging from this source has been eliminated.

2.—The date of raising the dam to its ultimate height is problematical and, no doubt, will be deferred for a considerable number of years, in which case the salvage value of radial gates would be comparatively small, and their remaining useful life would probably be too short to warrant their being re-installed in the spillway of the raised dam.

3.—Within recent years, the Engineering Profession has been giving attention to the scientific design of masonry dams and their appurtenances, and, by the time the Hetch Hetchy Dam is raised, it is possible that new methods of handling flood flows, either by more efficient spillway design or by means of large gates of the balanced needle type, will have been developed, in which case the radial gates would be valueless.

No doubt, a certain amount of vibration will be set up within the dam during the period of maximum flood discharge through the siphons, but, as Mr. Noetzli states, the large body of water back of the structure will have a damping effect, which in all probability will reduce the vibrations and the resulting increase of stresses in the dam, to a negligible amount.

Four years of careful observations of the Alpine Dam in Marin County, California, built under the writer's supervision as Consulting Engineer, disclose no serious effects from the operation of six siphon spillways in that structure. They are similar in type and design to the proposed construction at Hetch Hetchy. They cease operation without any violent shock. Owing to slight errors in construction, the two end siphons are 1 in. lower than the four central siphons and naturally operate before the latter become effective.

Floods behind this dam have never caused more than three of these siphons to come into action at one time. There is an advantage in breaking and aerating the overflow water curtain, thereby preventing any effects from vacuum.

The problem of preventing water under pressure from entering the joint between the present dam and its future addition was recognized as a serious one, and an attempt has been made to solve it, as shown on Plate X. The future construction joint between the new and the old parts of the dam has been treated in the same manner as a vertical contraction joint, that is, by placing a copper seal or water-stop across the joint and arranging it so that the copper can bend to accommodate any difference in deflections between the old and the new parts of the dam. This horizontal copper water-stop is connected to the vertical copper strip across the contraction joints, so that a complete seal is provided against water from the reservoir entering between the two parts of the work. When the dam is raised, provisions must also be made for thoroughly roughening the existing down-stream face and grouting the joint between the new and the old parts of the dam. A similar copper water seal has been provided, extending entirely around each siphon opening. The interior surfaces of the siphons will have to be thoroughly roughened with pneumatic chisels, etc., before filling them with new concrete, and provision made for grouting after the new concrete has set.

Measuring Deflections in Hetch Hetchy Dam

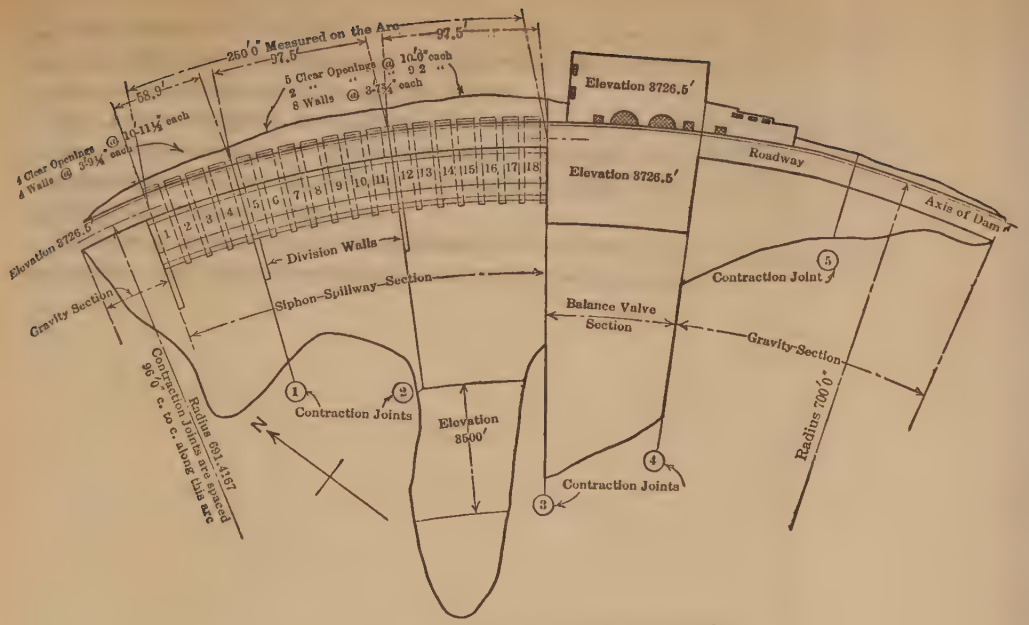
Arrangements are now being considered for installing centered bronze plugs or other devices in the upper half of the dam, so that deflections can be measured at various times under different water levels of the reservoir. These provisions were not made in the lower parts of the structure as, due to the great thickness and the fact that it is wedged into the canyon (having really a third abutment for a part of its length and height), the deflections will probably not be measurable, and if they were, they would not be representative of typical dams.

Effects of Ice on Lake Eleanor Dam

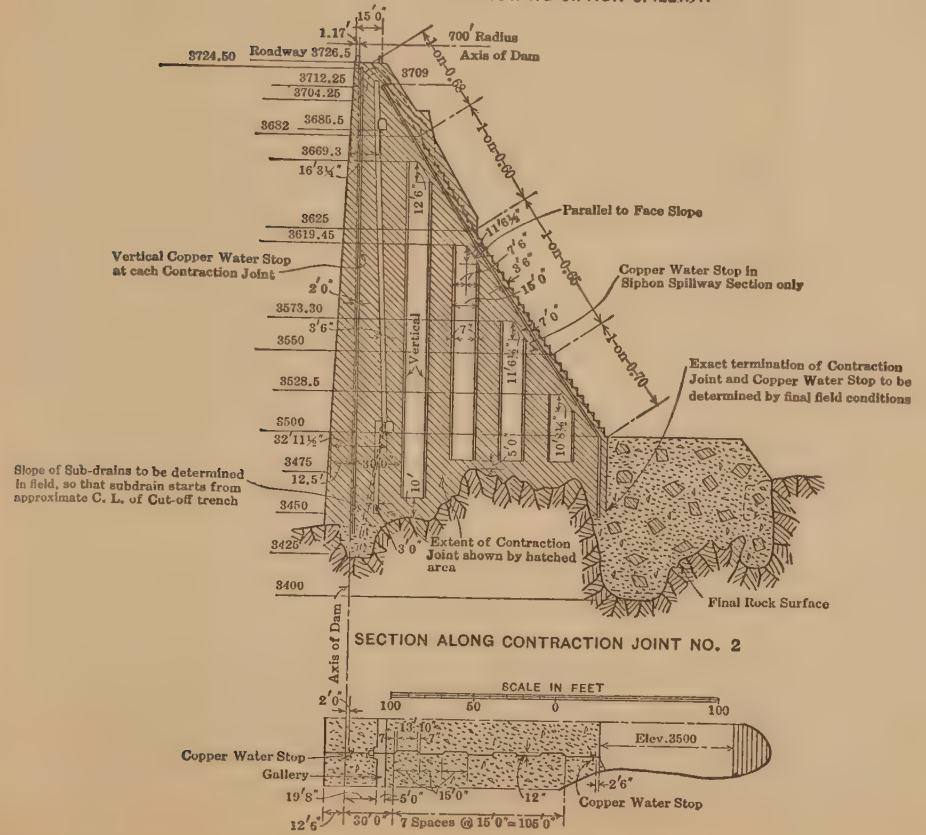
No harmful effects have been observed at the Lake Eleanor buttressed arch dam, 1 100 ft. higher in altitude than Hetch Hetchy, either from freezing and thawing or from ice pressure, the structure to date being practically bottle-tight. The winter period of ice formation occurs when the reservoir surface is at the lower stages where the arches are comparatively thick. Also, during this period, the reservoir is under steady draft, lowering the water surface at a rate which practically precludes the formation of large continuous thick ice sheets. Moreover, the arches are built on an incline, and any ice sheet which might form against the face of the dam, in expanding, would creep up the incline and, thus, to a great extent, relieve the pressure.

Porous Concrete Blocks for Drainage Wells

There is room for divergence of opinion as to how to make drainage wells in a dam most effective. When porous blocks are used, there is undoubtedly some clogging due to mortar from the surrounding concrete entering the voids

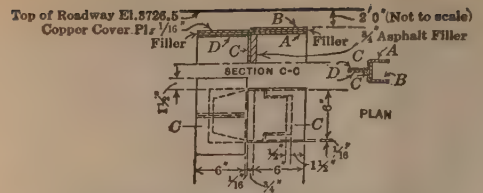


PLAN OF DAM SHOWING SIPHON SPILLWAY

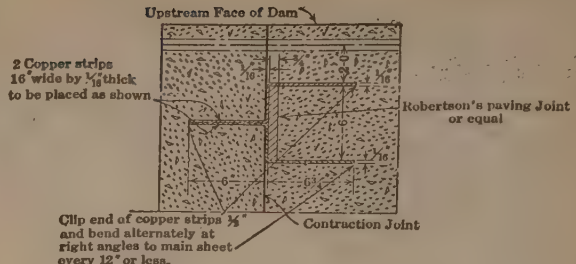

Roadway 3726.5
3724.50
700' Radius
Axis of Dam
Parallel to Face Slope
Vertical Copper Water Stop at each Contraction Joint
Copper Water Stop in Siphon Spillway Section only
Exact termination of Contraction Joint and Copper Water Stop to be determined by final field conditions
Slope of Sub-drains to be determined in field, so that subdrain starts from approximate C. L. of Cut-off trench
Extent of Contraction Joint shown by hatched area
Final Rock Surface
Axis of Dam
SCALE IN FEET
Copper Water Stop
Gallery
Elev. 3500
Copper Water Stop
7 Spaces @ 15'0" = 105'0"

SECTION ALONG CONTRACTION JOINT NO. 2

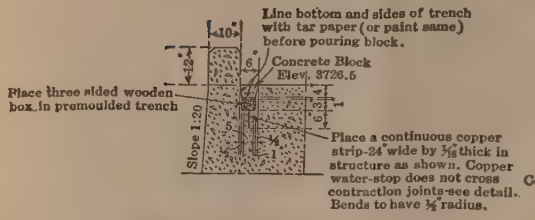
PLAN



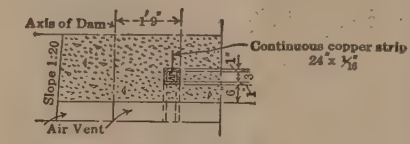
METHOD OF TERMINATING WATER-STOP


Upstream Face of Dam
2 Copper strips 16" wide by ⅛" thick to be placed as shown
Robertson's paving Joint or equal
Clip end of copper strips ⅓" and bend alternately at right angles to main sheet every 12" or less.
Contraction Joint

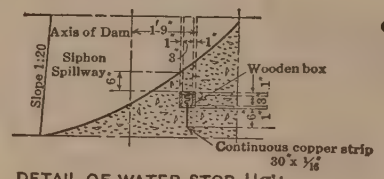
DETAIL OF COPPER WATER-STOP


Line bottom and sides of trench with tar paper (or paint same) before pouring block.
Concrete Block Elev. 3726.5
Place three sided wooden box in premoulded trench
Slope 1:20
Place a continuous copper strip 24" wide by ⅛" thick in structure as shown. Copper water-stop does not cross contraction joints-see detail. Bends to have ½" radius.

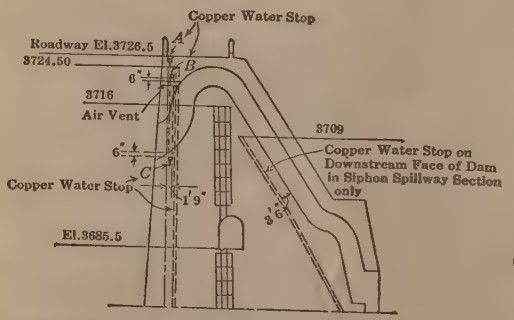
DETAIL OF WATER-STOP "A"



DETAIL OF WATER-STOP "B"

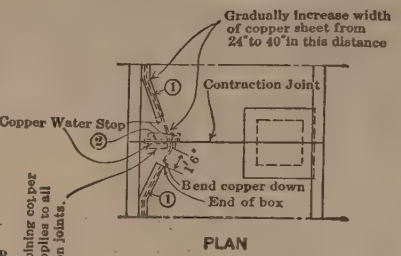


DETAIL OF WATER-STOP "C"

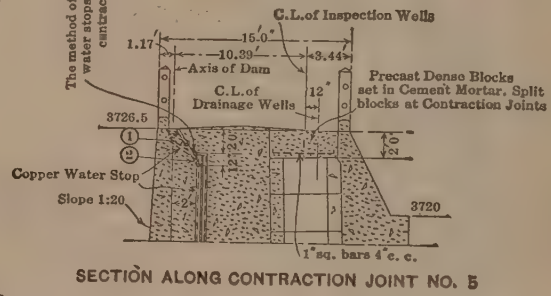


SECTION SHOWING LOCATION OF WATER-STOPS

SCALE IN FEET


Gradually increase width of copper sheet from 24" to 40" in this distance
Contraction Joint
Copper Water Stop
Bend copper down
End of box
The method of joining copper water stops applies to all contraction joints.

PLAN


C.L. of Inspection Wells
Axis of Dam
C.L. of Drainage Wells
Precast Dense Blocks set in Cement Mortar. Split blocks at Contraction Joints
3726.5
Copper Water Stop
Slope 1:20
3720
1" sq. bars 4" c. c.

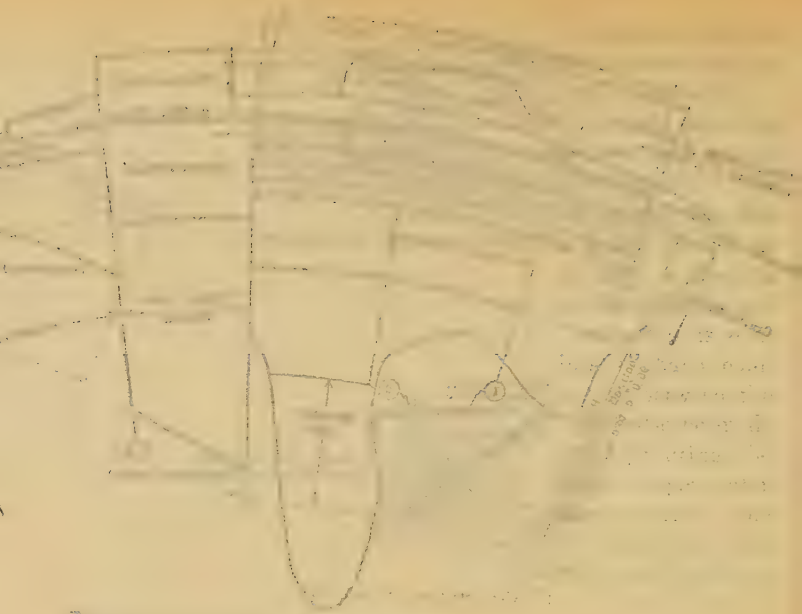
SECTION ALONG CONTRACTION JOINT NO. 5

HETCH HETCHY WATER SUPPLY OF THE CITY AND COUNTY OF SAN FRANCISCO, CALIFORNIA.

HETCH HETCHY DAM

SIPHON SPILLWAY PLAN, ELEVATION, SECTIONS CONTRACTION JOINT AND WATER STOP DETAILS

SCALE IN FEET



near the surface of the block; but the draining of this mortar from the new concrete prevents the formation of the relatively impervious surface, that results from placing concrete against forms.

Mr. Justin suggests using forms for drainage wells and breaking up the surface of the concrete by bush-hammering. To make this effective, it might be necessary to remove the surface concrete to a depth of ½ in. or more, which would be an expensive operation.

A test was recently made by digging out one block, and it was found that the mortar from the surrounding concrete did not penetrate ½ in.

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Paper No. 1496

THE CONTINUOUS TRUSS BRIDGE OVER THE OHIO RIVER AT SCIOTOVILLE, OHIO, OF THE CHESAPEAKE AND OHIO NORTHERN RAILWAY*

By Gustav Lindenthal,† M. Am. Soc. C. E.

With Discussion by Messrs. C. A. P. Turner, T. Kennard Thomson, Charles Evan Fowler, J. E. Greiner, D. B. Steinman, Henry H. Quimby, and Gustav Lindenthal.

Synopsis

The peculiar construction of the Sciotoville Bridge has been the subject of frequent inquiries, and the following detailed, although somewhat belated, description will serve as a permanent record, useful for similar bridge construction elsewhere.

The distinguishing features of the design are four: Continuous trusses over two long spans; floor-beams, acting as inverted arches and braced against tractive forces; erection with the minimum of falsework and without extra material in the trusses; and riveted connections to the limit of the largest rolling mill and shop facilities. The subject-matter is presented herewith under the following heads:

1.—General Conditions and Selection of Design.
2.—History and Characteristics of Continuous Truss Bridges.
3.—Substructure.
4.—Steel Superstructure.
5.—Fabrication and Erection of Steelwork.

1.—General Conditions and Selection of Design

Location and Grades.—When it was decided by the Chesapeake and Ohio Northern Railway Company to bridge the Ohio River and connect with the

* Presented at the meeting of April 5th, 1922.
† Cons. Engr., New York City.

main line of the Chesapeake and Ohio Railway, on the Kentucky side (the southern or left bank) of the Ohio River, the crossing was selected so as to form a direct route from the junction to Columbus, Ohio, along the valleys of the Little Scioto and the Scioto Rivers, with exceptionally favorable grades for the heavy coal transportation to the Great Lakes *via* the Hocking Valley Railway. Going north, the grades are 0.2% and, going south, 0.3%, and are compensated on curves. The line has a single track for the present, but the Ohio River Bridge and the masonry piers of the approach viaducts are built for two tracks. With the coal traffic steadily growing, two tracks for the entire line will be necessary before long. The double-track bridge was built at a period of low prices, thus enhancing its economic advantage in the development and growth of that important traffic.

River Conditions.—At the site of the bridge, the Ohio River has a width of about 1 600 ft. between embankments and forms a sharp bend. The river channel is near the inner or Kentucky shore, but at high-water stages, the traffic shifts toward the Ohio shore, along which also, at times, considerable ice and drift are carried.

The river channel is 5 ft. deep at low water and 72 ft. deep at high water. The bottom which is practically bare rock, with a slight slope from the Ohio toward the Kentucky shore, afforded a solid foundation for the piers.

The requirements of the War Department called for a minimum clear height under the bridge of 90 ft. above low water and 40 ft. above high water. The navigation interests demanded large openings, owing to the danger by obstructing piers to the descending coal tows some of which are 150 ft. wide and 700 ft. long and (at this sharp bend of the river, at the head of the shoals of the Little Scioto River) are difficult to control, particularly on account of the dense smoke and fog prevalent in this locality. On the Kentucky side, it was also necessary to keep the river channel open for navigation during the erection of the bridge.

Selection of Design.—After several layouts with different span lengths, it appeared that with two spans of 750 ft. each in the clear and a pier in the middle of the river, the requirements of the navigation interests and of the War Department would be satisfied. This made possible a symmetrical and sightly structure with two spans 775 ft., center to center, of bearings. (Fig. 1.)

The rock foundations were favorable to a continuous truss bridge, which also offered the advantage of erection with a minimum of falsework. Two simple truss spans of 775 ft. each, would have been from 15 to 20% more expensive for metal and erection.

The span of 775 ft. exceeds in length the longest existing simple span bridge, namely, the 720-ft. span of the Ohio River Bridge, at Metropolis, Ill., built in 1917.

It will thus be seen, that the selection of continuous trusses was primarily indicated in this case by reasons of economy in metal and by facilities of erection.

2.—History and Characteristics of Continuous Truss Bridges

In view of the fact that this bridge has the longest spans of the continuous truss type, and thus comes into competition, for long spans, particularly with

the simple span and the cantilever type, it appears appropriate to review briefly its history and some of its characteristics not generally appreciated by American bridge engineers.

History.—The Britannia Bridge in England, built by Stephenson in 1848, marks a milestone in bridge construction, not only because it was the first important iron bridge of the beam type, but also because it was the first representative of the continuous girder type. It is a tubular plate girder bridge of four spans, two of which are 230 ft. and two of 460 ft. The girders were proportioned as simple beams, but the designer, realizing that continuity increased the carrying capacity, regarded this feature as an additional safety. To gain more information, he made tests with a model, and thus found the points of contraflexure of the elastic line, which he regarded as "fixed" and as dividing the span into two "cantilevers" and a "central beam". This laid the foundation for the later development of the modern cantilever bridge by the introduction of hinges at the points of contraflexure.

Too much credit cannot be given to that galaxy of early English bridge engineers of nearly one hundred years ago—Stephenson, Fairbairn, Telford, Tierney Clark—for the originality and daring of their plans and constructions. They did their own thinking; they did not wait for precedents, but created them. Theirs was the genius that originates as distinguished from routine which merely imitates.

The Britannia Bridge was followed by several similar bridges, among which may be mentioned the Torksey Bridge, built in 1849, over the Trent, with two spans of 130 ft., and the Bryne Bridge, built in 1855, with a central span of 267 ft. and two side spans of 141 ft. each.

In the latter part of the Nineteenth Century, continuous truss bridges were extensively built on the Continent, principally in France, and were usually of the lattice truss type with parallel chords, with from three to five spans. The Fades Viaduct over the Sioule River, in France, built from 1905 to 1908, with a central span of 472 ft., was the longest continuous span previous to the building of the Sciotoville Bridge.

The Lachine Bridge, built in 1888, by the late C. Shaler Smith, M. Am. Soc. C. E., over the St. Lawrence River, near Montreal, Que., Canada, with two side spans of 269 ft., and two middle spans of 408 ft., was, for 29 years, the only continuous bridge in America. It was built as a cantilever and then converted into a continuous truss for the live load. It was replaced in 1910 by a simple span bridge.

There has been always more or less prejudice against continuous trusses, because cantilever trusses offer the alleged more accurate computation of stresses on purely statical principles. No continuous girder as far as known has ever failed in the trusses, whereas the largest and most discreditable bridge failure belongs to the supposedly accurate cantilever type.

It appears now, however, as if the continuous truss would again come into its own, for not only was it adopted for the Sciotoville Bridge, but also for the bridge of the Bessemer and Lake Erie Railroad built in 1918 over the Allegheny River near Pittsburgh, Pa., with spans of from 272 ft. to 520 ft., and for the bridge of the Hudson Bay Railway, across the Nelson River, built in 1918, with spans of 300 ft. and 400 ft.

FIG. 1.—SCIOTOVILLE BRIDGE OVER THE OHIO RIVER.

Characteristics of Continuous Bridges.—The continuous truss type has nowhere met with more indiscriminate and unqualified condemnation than by engineers in the United States, who have alleged three principal objections against it, none of which is novel or decisive:

1st.—That it is statically indeterminate, that is, its reactions and stresses are dependent on the elasticity of its members;

2d.—That it is subject to stresses from unequal settlements of its supports; and,

3d.—That it is affected by temperature changes.

Although these characteristics may be objectionable in certain cases, it can be readily shown that they are entirely unobjectionable in others, and they will be briefly discussed.

Static Indeterminateness

The argument that the stress computations are complicated can be readily dismissed. For preliminary and comparative designs, the simplifying assumption of constant moment of inertia of the vertical section through the girder or truss and the neglect of the elastic deformation of the web members, furnish sufficiently accurate and quick results. For the final design, the slight additional time and labor involved in the accurate calculation is of no account as continuous bridges will never be a common type.

The stress calculation, of course, is the more complicated the greater the number of spans, but, for other reasons, it is not advisable to have more than three or four spans in a continuous bridge.

The fact that the stresses are, to a certain extent, dependent on the elastic deformation of the members, is characteristic of all statically indeterminate structures, but in a properly designed continuous bridge the effect of even large variations in the elastic behavior is insignificant when compared with other uncertain stresses, such as secondary stresses, etc., which exist also in statically determinate structures.

Variation between the actual and the calculated elastic deformation may be due, first, to a difference between the assumed and the actual modulus of elasticity; and, second, to the influence of the details, such as gussets, splice and tie-plates, rivets, etc., which can only be roughly estimated.

A proportional change in the modulus of elasticity or in the sectional area of all members due to allowance for details has no influence on the reactions and stresses, but only on the deflections which change in the same proportion with the moduli.

The effect of non-proportional changes is well illustrated by a comparison of two sets of calculations made for the Sciotoville Bridge. It was planned that, during the various stages of erection, the trusses would pass successively through static conditions of a beam on two, three, four, and five supports (see Plate XII). The calculation of the reactions, for the purpose of determining the necessary jacking forces at the temporary supports, and of the deflections, for determining the jacking heights, were made, first, on the basis of the gross area of the main section of the members without allowance for details and,

second, on the basis of gross area of the main section plus the area of a section equivalent to 75% of the weight of the details. This additional area, due to details, varies for different members from 5 to 25% and averages about 20% of the gross area of the main section. For these two extreme assumptions, certain deflections differ by 25 to 60%, whereas the reactions differ by not more than 0.2% in the two-span condition, 465 ft.-775 ft., 10% in the three-span condition, 155 ft.-465 ft.-775 ft., 60% in the four-span condition, 155 ft.-155 ft.-465 ft.-775 ft., 11% in the three-span condition, 310 ft.-465 ft.-775 ft., and 0.7% in the final two-span condition, 775 ft.-775 ft.

The large differences in the reactions in the three and four-span conditions are due principally to the great variation in the length of the spans and to the comparatively great height of the short spans. Continuous trusses of great height and greatly different span lengths are, therefore, rightly objectionable, the more so, because they are also sensitive to settlements of the supports and to temperature changes. During erection, such a condition is of no consequence, because the reactions can be measured and the height of the bearings promptly adjusted if necessary. Where the spans are more nearly equal, and the trusses not unusually high in comparison with the span length, the effect on the reactions and stresses from a variation in the elasticity of the trusses is practically insignificant.

As a precaution, and to obviate any uncertainty of stress action, from dead load at least, it is always possible and advisable, as was done in the case of the Sciotoville Bridge, to measure certain reactions, and, if necessary, to adjust the height of the bearings until the reactions are correct.

In no case should the variation in span length in a continuous truss be so great that the live load on any span will cause a reversal of the dead load reactions of the adjoining span. For all these reasons, continuous trusses over several spans and on metallic towers, or on steel arches, should preferably be shallow in depth, a rule first practiced by French engineers.

It is interesting to note that the actual deflections of the Sciotoville trusses, as observed in the field, were nearly midway between the values computed under the two previously mentioned assumptions; in other words, an average addition of about 10% to the gross areas of the sections, to allow for details, should be made when calculating elastic deformations.

Effect of Settlements or Compressibility of Supports

Where considerable settlements of the foundations are to be anticipated, or where the supports are high elastic towers, the continuous type of bridge is not advisable, unless care is taken to eliminate the effect of inconstant levels by means of adjustable bearings.

The stresses caused by the ordinary compressibility of the supports can be computed and, if they are not unduly large, can be neglected or provision can be made in the sections.

Settlements of the foundations are less objectionable the longer the spans, as already mentioned. For similar continuous trusses with equal pro-

portion of height to span length, the same settlement of a support causes stresses approximately in inverse proportion to the span length. As settlements may be assumed as proportional to the foundation pressure and as the latter is about the same for long and short spans, under the same soil conditions, it follows that, in general, the danger of excessive stresses due to settlements is less the greater the span lengths. In special cases, it may be advisable to design the permanent details of the truss bearings on the piers and abutments so that they may be raised or lowered by hydraulic jacks at any time, as needed to maintain the original levels.

In the case of the Sciotoville Bridge, a settlement of 2 in. in one of the end piers, if it was possible, or a settlement of 1 in. in the middle pier, the others remaining undisturbed, would change the reactions by only 0.6 per cent. It is evident that even a considerably greater settlement, which would seriously disturb the vertical alignment of the track, would not objectionably affect the stress condition in the trusses. When the Kentucky end of the bridge was raised to its final position, after erection had been completed, difference in the jacking force for the last 3 in. of jacking was noticeable. This shows that for spans of this length continuous trusses are unobjectionable, even if the foundations do not rest on solid rock.

As already mentioned, the first continuous trusses had solid webs (Britannia Tunnel Bridge) and on the Continent small mesh lattice webs. The secondary stresses in the web are of no importance in the first type and, in the latter type, they may be considered rather beneficial, since they contribute to stiffness and absorb some of the bending stresses on the trusses, proven by the fact that the lattice can bear some load when the chords are cut away, only the chord sections over the bearings may need reinforcement against bending. In some instances, the continuous small mesh web girders over several spans were erected on falsework with a camber of $\frac{1}{300}$ to $\frac{1}{300}$ of the entire length of bridge and then let down on the pier bearings, to produce in the trusses initial bending stresses opposite those from live load. Such was the procedure of erection of the continuous truss viaduct of five spans on high iron towers over the Thur at Ossingen, Switzerland, in 1873. The writer was on the Engineering Staff for that structure, for which the statical computations were considered quite a feat at that time.

Temperature Effects

Temperature stresses in continuous trusses may be caused, first, by the expansion or contraction of intermediate supports, particularly high steel towers, and, second, by unequal temperature changes in different parts of the trusses themselves (uniform temperature changes in all members cause no stresses).

The first effect is similar to that of settlement of the piers and is greater, the higher the intermediate supports and the shorter the spans. In some existing bridges, it amounts to as much as 25% of the stresses from dead and live load. It is, however, insignificant and negligible in the case of long spans resting on shallow piers. As the stresses can be calculated, it is easy to make the necessary provision in the sections.

The second effect is neglected, as a rule, although in a case where the bottom chords are protected from the direct rays of the sun by a solid floor, the temperatures in the top and bottom chords may differ considerably. The effect of such a difference is similar to that of a variation in the elasticity of the truss members and may be serious in a case where the lengths of spans vary considerably.

In the case of the Sciotoville Bridge an average difference in temperature of 10° Fahr., between the top and bottom chord (the effect of the web members is insignificant), would change the end reactions by only 1.5%, which is negligible.

Summing up, one arrives at the well established conclusion that continuous trusses are generally unobjectionable under the following conditions:

1st.—Where the lengths of the spans do not vary greatly, and the trusses are not unusually high as compared with the span length.

2d.—Where the foundations rest on fairly solid ground, compressibility of the soil being less objectionable the longer the spans.

3d.—Where the intermediate supports are not excessively high in comparison with the span length.

In every continuous truss there are a few members, such as the chords near the points of contraflexure, or web members near points of maximum moment, which are most sensitive to the effects enumerated, because their sections, as required by the dead and live load stresses, are comparatively small. Such members, as a rule, will require for fabrication or erection purposes a section somewhat in excess of that required by the stresses, to provide a certain margin for possible variation. It is advisable always to investigate such members and proportion them so that they are strong enough under reasonably extreme assumptions.

Advantages of Continuous Trusses.—Against the disadvantages mentioned, as far as they can be classed as such in any given case, must be weighed the advantages which the continuous type possesses over the simple span or the cantilever.

As regards economy, it is not feasible to make a general comparison between the continuous bridge and the cantilever since that depends largely on the arrangement of the spans, which is usually governed by local conditions and of necessity must be different for the two types, owing to their different character. In comparison with a series of simple spans of the same length, however, the continuous truss shows a decided economy which is greater the longer the spans and, up to a certain limit, greater the number of spans. For long spans, the saving in cost may be as much as 25 per cent. For short spans, the economy over simple spans is not important, but greater rigidity under passing loads is an advantage.

In point of rigidity, as measured by the deflections, the continuous truss compares favorably with the simple span and shows a decided advantage over the cantilever. Its deflections and amplitude of vibrations are smaller, and the elastic line smoother and devoid of local kinks such as occur at the hinges of cantilevers. Both rigidity and economy gain from the fact that

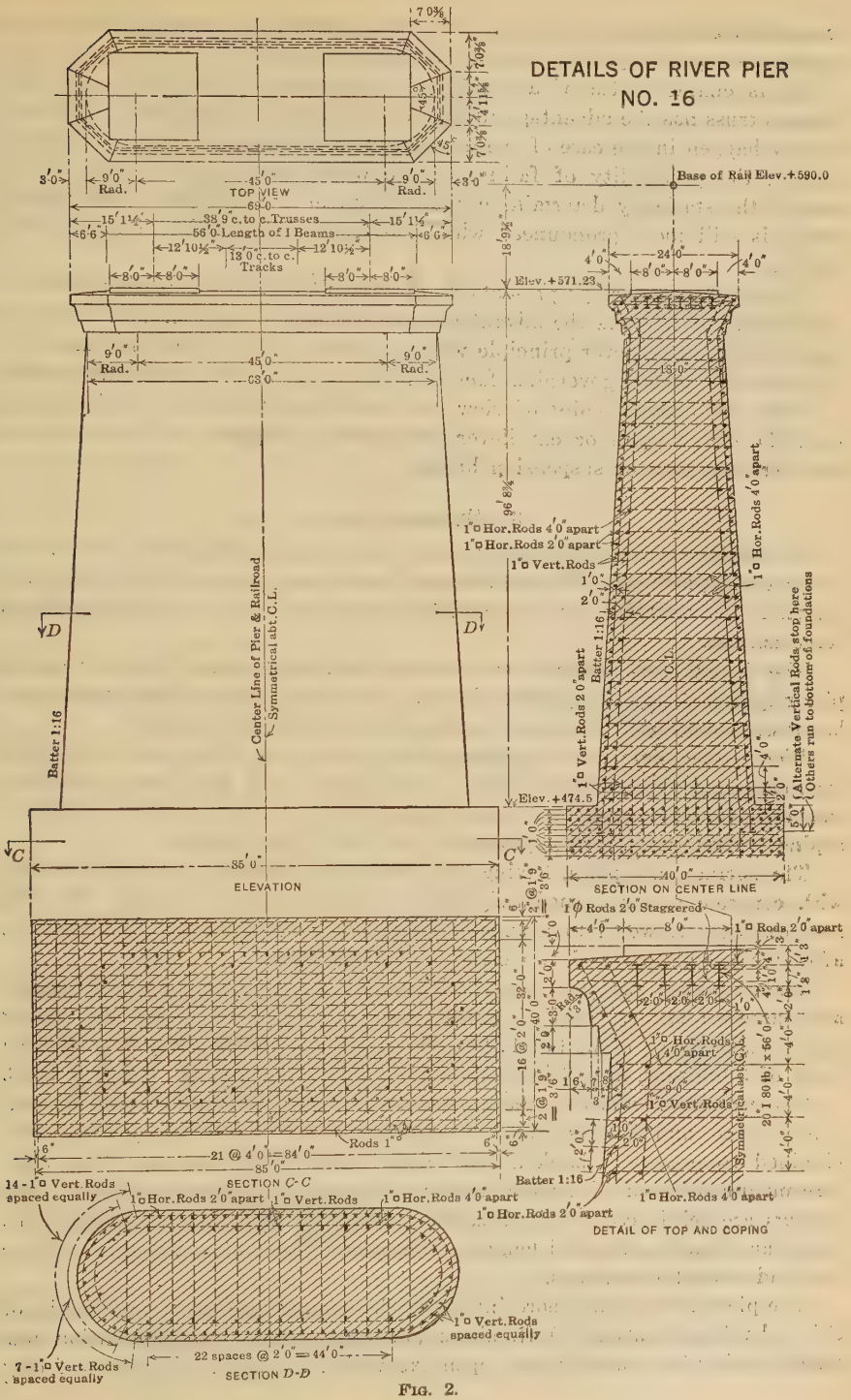

DETAILS OF RIVER PIER NO. 16
TOP VIEW
ELEVATION
SECTION ON CENTER LINE
SECTION C-C
SECTION D-D
DETAIL OF TOP AND COPING
Base of Rail Elev. +590.0
Elev. +571.23
Elev. +474.5
Batter 1:16
Center Line of Pier & Railroad
Symmetrical abt. C.L.
1" □ Hor. Rods 4'0" apart
1" □ Hor. Rods 2'0" apart
1" □ Vert. Rods
1" □ Rods 2'0" Staggered
(Alternate Vertical Rods stop here
(Others run to bottom of foundations
14 - 1" □ Vert. Rods spaced equally
7 - 1" □ Vert. Rods spaced equally
22 spaces @ 2'0" = 44'0"
21 @ 4'0" = 84'0"
85'0"
69'0"
38 9'c. to c. Trusses
56'0 Length of I Beams
18'0 c. to c. Tracks

FIG. 2.

truss members subject to reversion of stress (tension and compression) can be riveted members proportioned only for the larger stress.

As is characteristic of all statically indeterminate structures, the continuous truss has the advantage in that when a member is seriously weakened, as may happen in the case of the derailment of a train and collision with the trusses, the probability of failure of the whole bridge is less than in the case of the statically determinate simple span or a cantilever. This greater safety is still more pronounced when the truss webs consist of small mesh lattice.

The continuous bridge presents no greater difficulties in erection than the cantilever, but it offers the advantage over the simple span in that it can be erected on the cantilever principle without, or with little, additional material. This was one of the governing factors in the case of the Sciotoville Bridge.

From the esthetic point of view, the continuous bridge can well compete with the simple span or cantilever, if properly designed, but not with the more artistic arch or suspension bridge.

3.—Substructure

Foundations.—All the piers rest on solid shale rock which has nearly vertical stratification. The maximum foundation pressure is 9.5 tons per sq. ft. from vertical loads only and 12.5 tons per sq. ft., with longitudinal force acting.

The foundations presented no unusual problems, except that the center pier required a very heavy coffer-dam to obviate the danger of the coffer-dam being washed away by flood, as it had to rest on bare rock bottom.

Piers.—The piers are of concrete, reinforced with steel rods to prevent shrinkage and temperature cracks. Both the up and down-stream ends of the piers are semi-circular. The tops are marked by octagonally shaped massive copings which give a neat appearance at no additional expense.

The surfaces are plain, having been wetted and rubbed to a smooth finish with concrete bricks immediately after the forms were removed. The top surfaces were troweled to a smooth finish before the concrete had set and, after hardening, received two coats of cement filler.

The center pier which resists the longitudinal force from the entire bridge and carries a vertical load of 16 400 tons, is 18 by 63 ft. under the coping and has a batter all around of 1:10. The shore piers which carry only vertical loads of 5 100 tons, are 12 by 57 ft. under the coping and are battered 1:20.

The concrete is mixed in the proportion of 1 part Lehigh Portland cement, 2 parts sand, and 4 parts gravel. Sand and gravel of excellent quality were obtained from a river bank a few miles below the bridge site. The reinforcement consists of 1-in. square steel rods, arranged as shown on Fig. 2. The copings are reinforced longitudinally by 20-in. by 80-lb. beams, spaced 2 ft. apart, and running the full length of the pier. They serve also to distribute the load from the bearings.

The pier on the Ohio shore intersects the ground surface at mid-height of the embankment, about 30 ft. above low-water level. The foundation was placed during low water by dry, open excavation through about 10 ft. of clay

and 8 ft. of loose disintegrated shale rock. This pier was started in November, 1914, and completed in March, 1915.

The pier on the Kentucky shore is near the bottom of the embankment, where the ground surface is only about 15 ft. above low-water level and the soil is mostly sand. These conditions made it desirable to sink the pier as a concrete caisson with a steel cutting edge, a working chamber 6 ft. high, and four shafts 7 ft. 3 in. in diameter. It was not necessary, however, to resort to air pressure. All excavation down to the solid rock, about 15 ft. below low water, was done by open dredging with orange-peel buckets while the caisson was flooded. About 2 ft. of disintegrated rock was removed by hand, for which purpose the caisson was kept dry by pumping, having been carefully sealed with empty cement bags rolled up behind the cutting edge. As there was considerable seepage through the rock bottom, drain sumps were provided from which the water was pumped during the concreting of the working chamber. These sumps were finally filled by forcing cement grout down through pipes by compressed air. The cutting edge was set on June 22d, and the pier completed on September 14th, 1915.

At the middle or river pier, the rock surface is practically level and barely exposed at low water. The rock was excavated to a depth of 10 ft. to give the pier a firm hold against dislocation under possible ice pressure. This depth also proved necessary for the removal of all disintegrated and seamy rock.

Excavation and concreting were carried on within a wooden coffer-dam set on the rock bottom. This coffer-dam was 16 ft. high, 81 ft. wide, and 129 ft. long, and had double walls 10 ft. apart. The space between the walls was filled with dirt dredged from the river channel. The top was capped with 2-in. planking to prevent the washing out of the fill. Beams 8 ft. high were placed outside and inside, along the dam. Although submerged for four months during the high-water season, the coffer-dam remained intact.

Construction of the coffer-dam was started in November, 1914, and the pier was completed in May, 1915, after an interruption of four months in the foundation work during high water.

The three piers contain approximately 15 000 cu. yd. of concrete and 250 tons of steel reinforcement and cost $165 000, or $11 per cu. yd. They were built on contract by the Dravo Contracting Company, of Pittsburgh, Pa., with an average daily force of 105 men.

4.—Steel Superstructure

Rigidity was one of the main considerations in working out the design of the steel superstructure. With a few modifications, the rules of design were the same as those for the Hell Gate Arch Bridge and Approaches, and are contained in detail, with explanatory remarks, in the paper by O. H. Ammann, M. Am. Soc. C. E., on that bridge.* They are, therefore, only briefly quoted here insofar as they apply specifically to the Sciotoville Bridge.

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 852.

Loads and Stresses.—All stresses except those from wind and lateral forces were computed by the accurate theory of elasticity, considering the elastic deformation of all members after their section had been determined. The stresses from the wind and lateral force were computed by assuming a constant moment of inertia of the lateral truss. The assumed loads were as follows:

The dead load (D) was carefully calculated for each panel point after the general details had been fully worked out. A re-calculation of the weight from the shop drawings showed the assumed weight to be 3% in excess and, therefore, no re-calculation was made. The average dead load is 18 200 lb. per lin. ft. of bridge, which includes 1 400 lb. per lin. ft. assumed for the two tracks.

The live load (L) on each track was as shown on Fig. 3 (Cooper's E-60 loading).

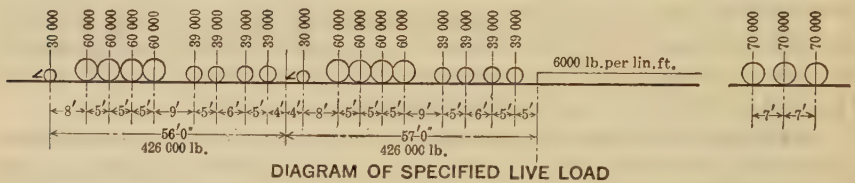


DIAGRAM OF SPECIFIED LIVE LOAD

FIG. 3.

In cases where the live load had to be separated, in order to produce maximum stress, the engine load was placed on the stretch producing the greater portion of the stress and a uniform load of 6 000 lb. per ft. of track on the stretch contributing the smaller portion.

The impact (I) was calculated according to the writer's formula, as follows:

$$I = \frac{L}{D + L} \times \frac{1\,200 + \frac{a}{n}}{600 + 4\,a}$$

in which,

D = dead load stress, in pounds.
L = live load stress, in pounds.
a = length of train behind locomotive tender for position of maximum stress, in feet.
n = number of tracks loaded for maximum stress.

The lateral force (Lat.) from trains was 10% of the live load on one track, or Cooper's E-6 loading acting horizontally 5 ft. above the rail.

The wind pressure (W) was 800 lb. per lin. ft., stationary along the top chord, 700 lb. per lin. ft., stationary along the bottom chord, and 500 lb. per lin. ft., moving 7 ft. above the rail.

The longitudinal force (Br.) from braking or traction on one track only was 60 000 lb. on a wheel-base of 15 ft. for each locomotive, or 1 000 lb. per lin. ft. of track for the whole train.

The total stress was as follows: Members with dead and live load stress were proportioned for a total stress of $D + L + I$ + Lat. + Excess, in which

Excess = (W and Br.) — 20% ($D + L + I$ + Lat.). Members free from dead or live load stress were proportioned for a total stress of W + Br. + Lat.

The permissible unit stresses, in pounds per square inch, were as follows:

Tension, net section	20 000
Compression, net section	20 000

Gross section, $20\,000 - 100\left(\frac{l}{r} - 20\right)$ rounded up to the nearest 500 lb., in which l = the length and r = the radius of gyration of the whole section.

For latticed members, a further deduction of $100\left(\frac{l_1}{r_1} - 20\right)$ was made, in which l_1 = the length and r_1 = the radius of gyration of the unsupported portion of the section between lattice connections.

Bending on rolled shapes, girders, and steel castings, net section	20 000
Bending on pins	30 000
Shearing on webs, shop rivets, and pins	15 000
Shearing on field rivets and bolts	12 000
Bearing on shop rivets	25 000
Bearing on field rivets, turning-bolts and pins	20 000
Pressure on concrete masonry	600

The specification for proportioning compression members differs somewhat from that used in the Hell Gate Bridge. The specified compression stress is applied to the gross instead of the net area of the section with the provision, however, that the stress applied to the net area shall not exceed 20 000 lb. per sq. in. The net area usually governed for members with $\frac{l}{r}$ less than 50.

Further, instead of different sets of unit stresses for different types of sections (closed section and sections with one or two open sides), the same basic stress was used for all sections, subject, however, to the previously stated deduction for latticed members.

General Proportions.—The steel superstructure, as built, is a double-track, two-span, continuous bridge, with a total length of 1 550 ft. between centers of end piers, or two equal spans of 775 ft. and two clear openings of 750 ft. at low-water level.

To obtain ample lateral rigidity, the width between centers of trusses was made 38 ft. 9 in., or one-twentieth of the span length, although at first a somewhat smaller width was considered sufficient, in view of the continuity of the lateral truss. The height at the middle of each span is 103 ft. 4 in. between centers of chords. To fix this height, the portion of the span between the end pier and the point of contraflexure was considered as a simple span. This portion varies from three-fourths of the span length for uniform load on both spans to seven-eighths of the span length for uniform load on one span only and averages 630 ft. in length. The height of the truss was chosen approximately one-sixth of this length.

The height over the center pier is 129 ft. 2 in., or one-sixth of the span length; this is the proper height for a simple span of the same length, which

has the same maximum moment at the center as the two-span continuous bridge over the middle support.

The height at the end was made 77 ft. 6 in., or equal to a double panel, in order to give the end posts an inclination of not less than 45 degrees. These heights also secured a pleasing outline for the top chord. The web system is of the Warren type, with subdivided panels of a uniform length of 38 ft. 9 in., which was found to be the most economical.

Truss Members and Connections.—Two preliminary designs of the trusses were made, one with tension members built up of pin-connected, 16-in. eye-bars and one with riveted members and riveted connections throughout. In the first design, all riveted members had riveted connections as most of them had to be designed for reversal of stress and no pin connections were allowed for such members. This design was unusual, in that the eye-bars in the heavier tension members were arranged in two chains, one above the other, each consisting of two sets of bars corresponding to the two webs or gussets of the riveted members.

The principal panel points were carefully detailed for both designs, and it was found that they were feasible, although requiring unusual connections and gussets of the largest practicable sizes.

Bids were asked on both designs. The eye-bar design, although about 200 tons lighter, proved to be only slightly cheaper, according to the lowest bid. The riveted truss design was finally adopted, in view of its superior rigidity, durability, and safety. The Sciotoville Bridge is, therefore, the largest truss bridge with completely riveted connections.

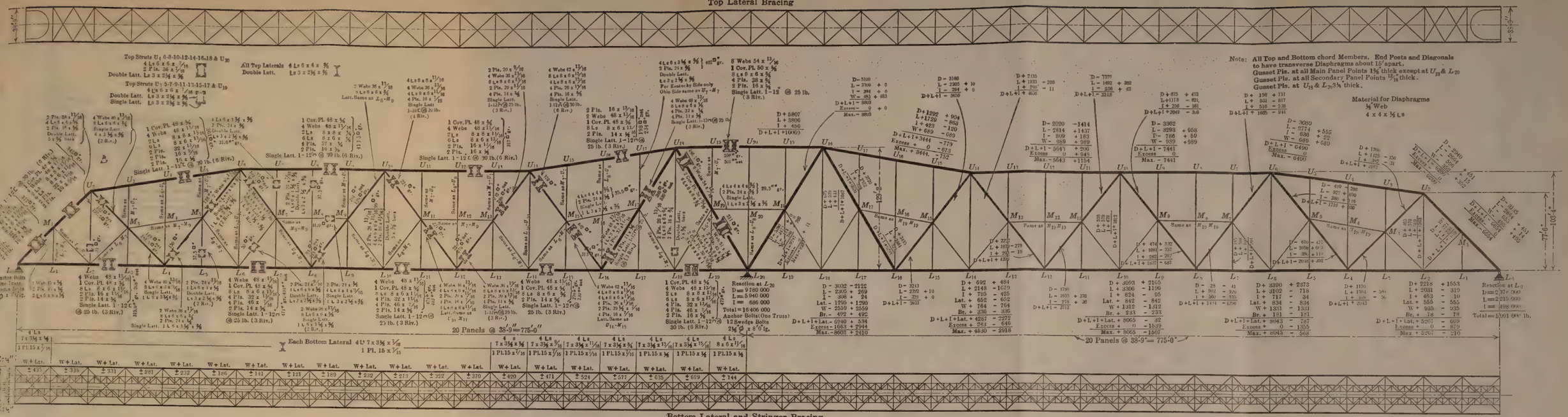
The advantages of pin-connected bridges, namely, cheaper fabrication and quicker erection, are not as important to-day as they were formerly. With the present improved facilities for punching, drilling, riveting, etc., the cost of manufacture and erection of riveted work has been greatly decreased, and the time of erection is a less important factor than in the pioneer days of rapid railroad construction.

Typical sections of the truss members are shown on the stress sheet (Plate XI). All members have double webs and all chords and main diagonals have inside and outside flange angles. Both top and bottom chords and the inclined end posts have one solid cover-plate on top. The bottom flange angles of the same web are connected by a flange plate which distributes the stress from the latticing to both angles. The inclined posts at the center pier have a solid cover, top and bottom, and thus form completely closed boxes.

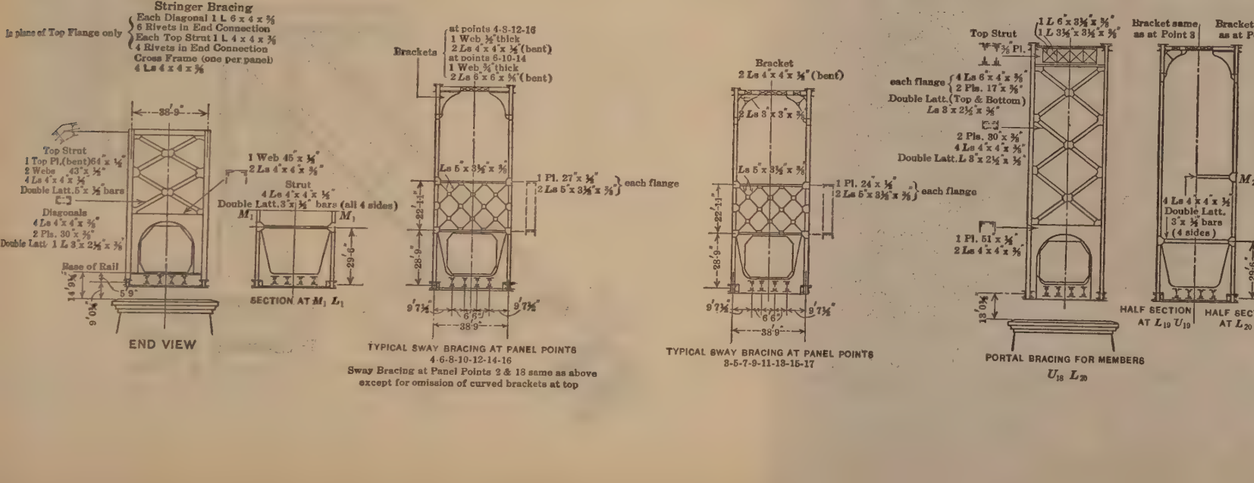
All the open sides of the members have exceptionally strong latticing, ranging from 3 by $2\frac{1}{2}$-in. by $\frac{3}{8}$-in. angles, with two rivets, to 12-in. by 30-lb. channels with six rivets. For compression members, the latticing has been proportioned for a transverse shearing force, in pounds, equal to three hundred times the gross area of the member, in square inches. The members are stiffened against distortion by transverse diaphragms about 15 ft. apart.

The gusset-plates are in the plane of the web-plates of the members, the latter being cut at the edge of and spliced to the gussets in such a manner that the rivets are in double shear. The flange angles extend in all cases over the gussets.

Top Lateral Bracing


Top Struts U1 6-8-10-12-14-16-18 & U20
Top Struts U1 5-7-9-11-13-15-17 & U19
All Top Laterals
Double Latt.
Single Latt.
Note: All Top and Bottom chord Members, End Posts and Diagonals to have transverse Diaphragms about 15' apart.
Gusset Pls. at all Main Panel Points 1⅝" thick except at U18 & L20
Gusset Pls. at all Secondary Panel Points 1⅝" thick.
Gusset Pls. at U18 & L20 1¾" thick.
Material for Diaphragms
⅜ Web
4 x 4 x ⅜ Ls
20 Panels @ 38'-9" = 775'-0"
Each Bottom Lateral
1 Pl. 15 x 7/16
W+ Lat.

Bottom Lateral and Stringer Bracing


Stringer Bracing
Each Diagonal 1 L 6 x 4 x ⅜
6 Rivets in End Connection
Each Top Strut 1 L 4 x 4 x ⅜
4 Rivets in End Connection
Cross Frame (one per panel)
4 Ls 4 x 4 x ⅜
Top Strut
Diagonals
Rose of Rail
END VIEW
SECTION AT M1 L1
Brackets
Bracket
TYPICAL SWAY BRACING AT PANEL POINTS
4-6-8-10-12-14-16
Sway Bracing at Panel Points 2 & 18 same as above
except for omission of curved brackets at top
TYPICAL SWAY BRACING AT PANEL POINTS
3-5-7-9-11-13-15-17
Top Strut
each flange
Double Latt.
Bracket same as at Point 3
Bracket same as at Point 4
PORTAL BRACING FOR MEMBERS
U18 L20
HALF SECTION AT L19 U19
HALF SECTION AT L20 U20

Assumed Dead Load Panel Concentrations per Truss in Units of 1000 lb.

Panel Points	Sum of Concentrations for Half Truss	20	19	18	17	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
Steelwork at Upper Chord	2021.3	63.5	111	300	53.5	61.9	53	135	65.4	80	68.4	159.5	83.7	96.4	84	158.3	77.5	90	78	149.2	0	0
Steelwork at Middle Points	1154.6	11	219.6	0	148.5	24.4	132.3	0	102.6	22.9	89.6	0	54.3	22.6	49.8	0	59.5	21.2	66	0	130	0
Steelwork at Lower Chord	3552.2	176	159.2	199	188	251.7	132.7	168	133	225.1	157	196.5	157	200	157.2	196.7	167	187.3	126	148.8	125	146.5
Total Steelwork	6728	250.3	484	499	360	338	318	353	304	328	315	356	295	319	291	355	294	299	270	298	255	146.5
Flooring	540	13.5	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	13.5
Total Steelwork and Flooring	7268	264	511	526	387	365	345	380	331	355	342	383	322	346	318	382	321	326	297	325	282	160
Bearings																						

General Notes:
All Stresses are given in Units of 1000 lb.
D. denotes Dead Load
L. " Live Load
I. " Impact
Lat. " Lateral force
W. " Wind
Br. " Braking
Excess = (W + Br.) − 20% (D + L + I + Lat.)

$$I = \frac{L^2}{D+L} \times \frac{1200 + \frac{n}{m}}{600 + 4\alpha}$$

α = length in ft. of train behind Locomotive tender for position causing max. Stress
n = number of tracks

All field connections of stringers to floorbeams shall not be riveted until span is completely erected and self supporting

Size of Rivets

Trusses:
- All main members { shop = 1" dia. { except at Points U18 L20 = 1⅛" dia.; field = 1" dia.
- Sub diagonals, main & sub hangers { shop = ⅞" dia.; field = ⅞" "
- Long secondary posts { shop = ⅞" dia. { except in connection for Bott. Chord Gussets = 1" dia.; field = ⅞" "

Floorsystem:
- For balance of truss members & bracing { shop = ⅞"; field = ⅞"
- shop = ⅞" dia. {except for the end connections field = ⅞" dia. } of Floorbeams and stringers. = 1" dia.

Computation of Stresses

All stresses determined by assuming main trusses and bottom lateral truss as continuous over three supports and top lateral trusses as simple spans between U_2 and U_{13}

STRESS AND SECTION SHEET

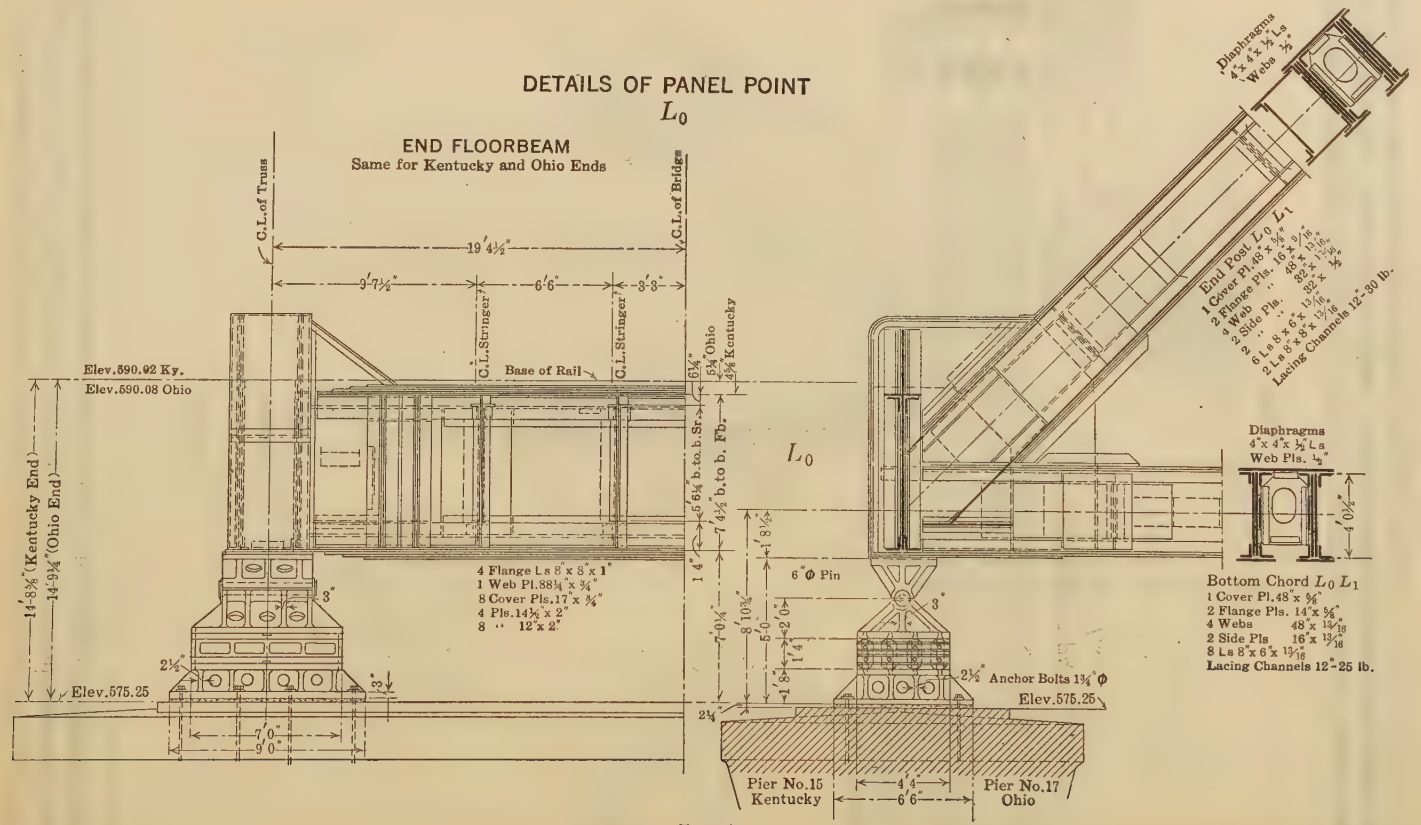

DETAILS OF PANEL POINT
L_0
END FLOORBEAM
Same for Kentucky and Ohio Ends
C.L. of Truss
C.L. of Bridge
19′4½″
9′7½″
6′6″
3′3″
C.L. Stringer
C.L. Stringer
Base of Rail
Elev. 590.92 Ky.
Elev. 590.08 Ohio
14′8⅜″ (Kentucky End)
14′9¼″ (Ohio End)
Elev. 575.25
6¼″
5¾″ Ohio
4⅝″ Kentucky
6′6¾″ b. to b. Sr.
7′4½″ b. to b. Fb.
1′8½″
1′4″
7′0¼″
8′10¾″
5′0″
2′0″
1′4″
1′8″
2¼″
4 Flange Ls 8″ x 8″ x 1″
1 Web Pl. 88¾″ x ¾″
8 Cover Pls. 17″ x ⅝″
4 Pls. 14½″ x 2″
8 " 12″ x 2″
3″
2½″
7′0″
9′0″
L_0
6″ Φ Pin
2½″ Anchor Bolts 1¾″ Φ
Elev. 575.25
Pier No. 15
Kentucky
4′4″
6′6″
Pier No. 17
Ohio
Diaphragms
4″ x 4″ x ¾″ Ls
Web ½″
End Post L_0 L_1
1 Cover Pl. 48″ x ⅝″
2 Flange Pls. 16″ x 9/16″
2 Web Pls. 48″ x 13/16″
2 Side Pls. 32″ x 13/16″
" 32″ x ½″
6 Ls 8″ x 6″ x 13/16″
2 Ls 8″ x 8″ x ½″
Lacing Channels 12″-30 lb.
Diaphragms
4″ x 4″ x ½″ Ls
Web Pls. ½″
10½″
Bottom Chord L_0 L_1
1 Cover Pl. 48″ x ⅝″
2 Flange Pls. 14″ x ⅝″
4 Webs 48″ x 13/16″
2 Side Pls 16″ x 13/16″
8 Ls 8″ x 6″ x 13/16″
Lacing Channels 12″-25 lb.

FIG. 4.

The gusset-plates at the main panel points are $1\frac{5}{8}$ in. thick, except at panel points, U_{18} and L_{20}, where they have a maximum thickness of $3\frac{1}{4}$ in. The gussets at the secondary panel points are $\frac{13}{16}$ in. thick. Correspondingly,

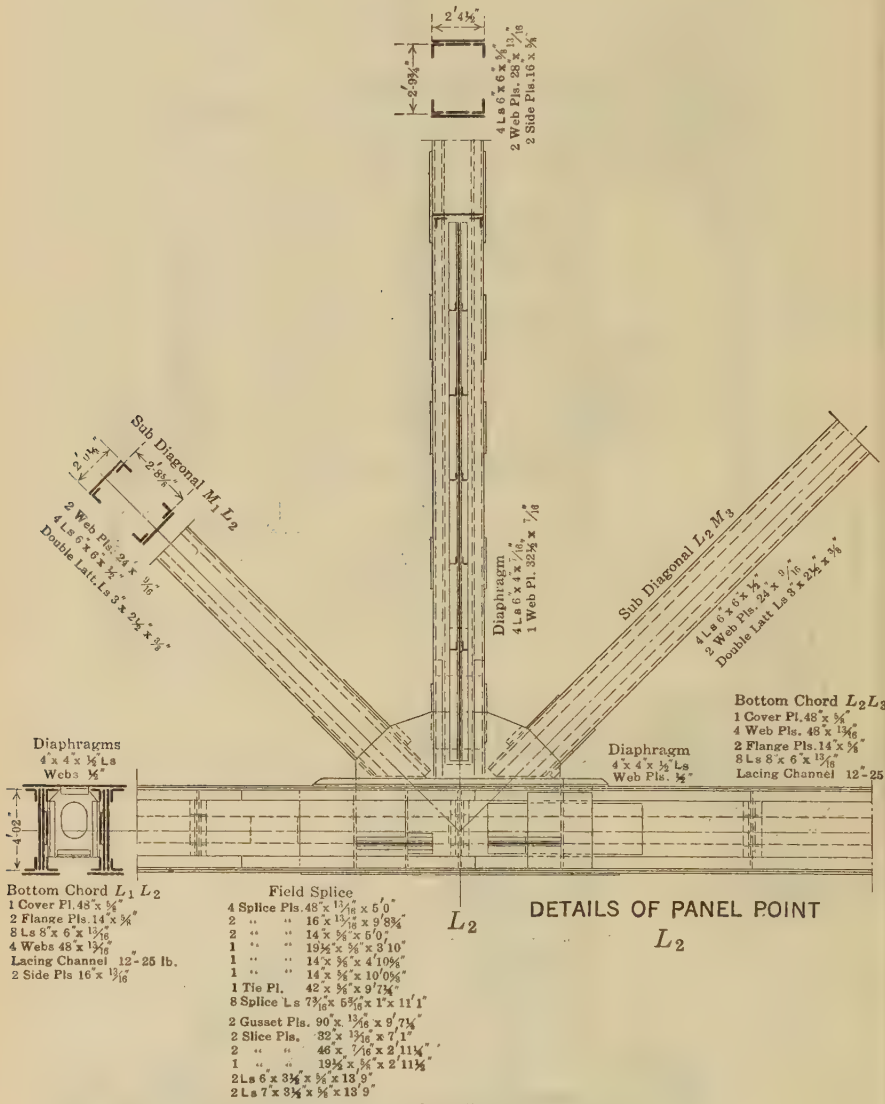
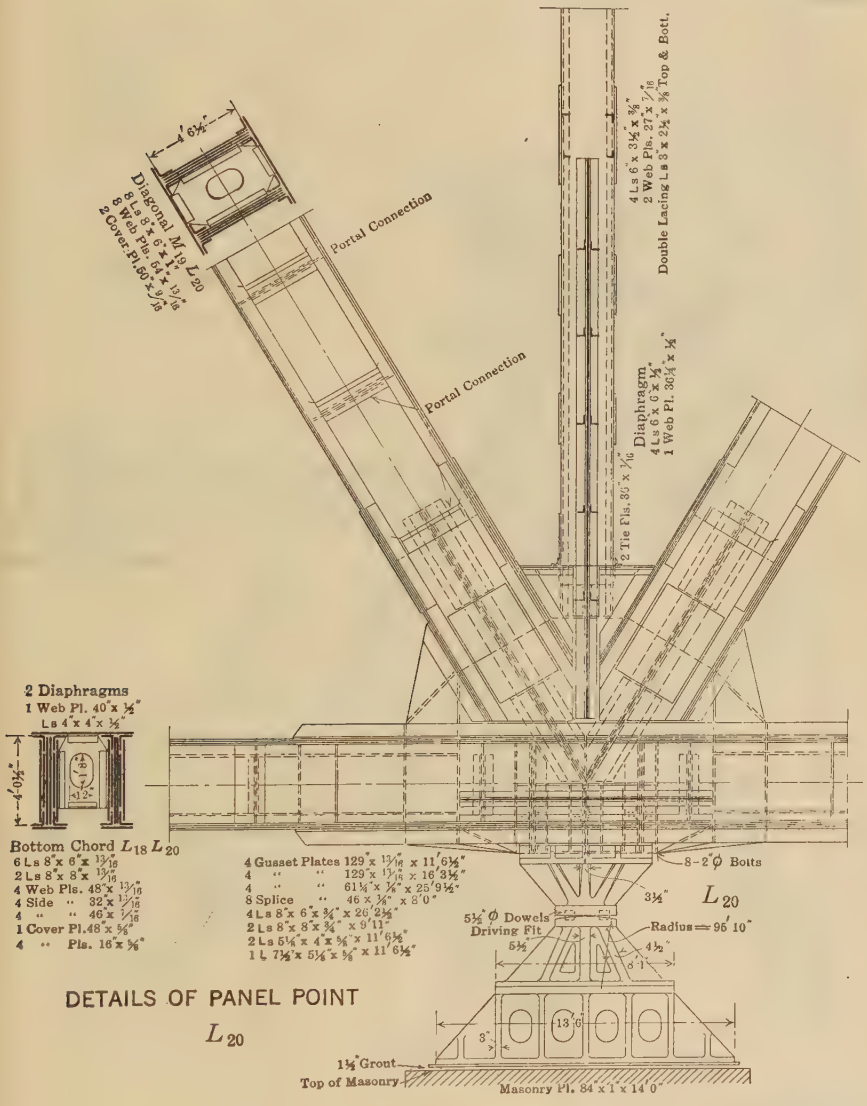


FIG. 5.

the webs of the main members are $\frac{13}{16}$, $1\frac{5}{8}$, and $3\frac{1}{4}$ in. thick, all being made up of individual plates, $\frac{13}{16}$ in. thick. This uniformity greatly facilitated the detailing of the connections and the avoiding of fillers.

As the full thickness of the gussets was not required for the full size, at some of the largest connections the gussets were split into two to four plates, $\frac{13}{16}$ in. thick, of variable size, in order to save weight. The largest gusset-plates used are 135 in. by $1\frac{5}{8}$ in. by 14 ft. 9 in., and 140 in. by $\frac{13}{16}$ in. by 18 ft. 2 in.


Portal Connection
Portal Connection
Diagonal M_{18} L_{20}
8 Ls 8" x 6" x 1 1/16"
8 Web Pls. 64" x 13/16"
2 Cover Pl. 80" x 9/16"
4 Ls 6" x 3½" x ⅜"
2 Web Pls. 27" x 7/16"
Double Lacing Ls 3" x 2¼" x ⅜" Top & Bott.
Diaphragm
4 Ls 6" x 6" x ½"
1 Web Pl. 36¾" x ¼"
2 Tie Pls. 36" x 7/16"
2 Diaphragms
1 Web Pl. 40" x ½"
Ls 4" x 4" x ½"
Bottom Chord L_{18} L_{20}
6 Ls 8" x 6" x 13/16"
2 Ls 8" x 8" x 13/16"
4 Web Pls. 48" x 13/16"
4 Side " 32" x 13/16"
4 " " 46" x 13/16"
1 Cover Pl. 48" x ⅝"
4 " Pls. 16" x ⅝"
4 Gusset Plates 129" x 13/16" x 11' 6½"
4 " " 129" x 13/16" x 16' 3½"
4 " " 61⅛" x ⅞" x 25' 9½"
8 Splice " 46" x ⅞" x 3' 0"
4 Ls 8" x 6" x ¾" x 26' 2½"
2 Ls 8" x 8" x ¾" x 9' 11"
2 Ls 5¼" x 4" x ⅝" x 11' 6½"
1 L 7½" x 5¼" x ⅝" x 11' 6½"
8-2" ϕ Bolts
3½"
L_{20}
5½" ϕ Dowels
Driving Fit
Radius = 95' 10"
5½"
4½"
13' 6"
8"
1¼" Grout
Top of Masonry
Masonry Pl. 84" x 1" x 14' 0"

DETAILS OF PANEL POINT
L_{20}

FIG. 6.

All chords are fully spliced. In the Ohio span, which was erected on falsework, the splices are arranged at every second panel point, and on the Kentucky side at every panel point for convenience in the cantilever erection.

All shop and field rivets of the main truss members are 1 in. in diameter, except at panel point, U_{18} and L_{20}, where they are $1\frac{1}{4}$ in. in diameter and up to $7\frac{7}{8}$ in. long between heads. The secondary members have $\frac{7}{8}$-in. shop and 1-in. field rivets. Figs. 4, 5, 6, 7, and 8 show typical connections and splices.

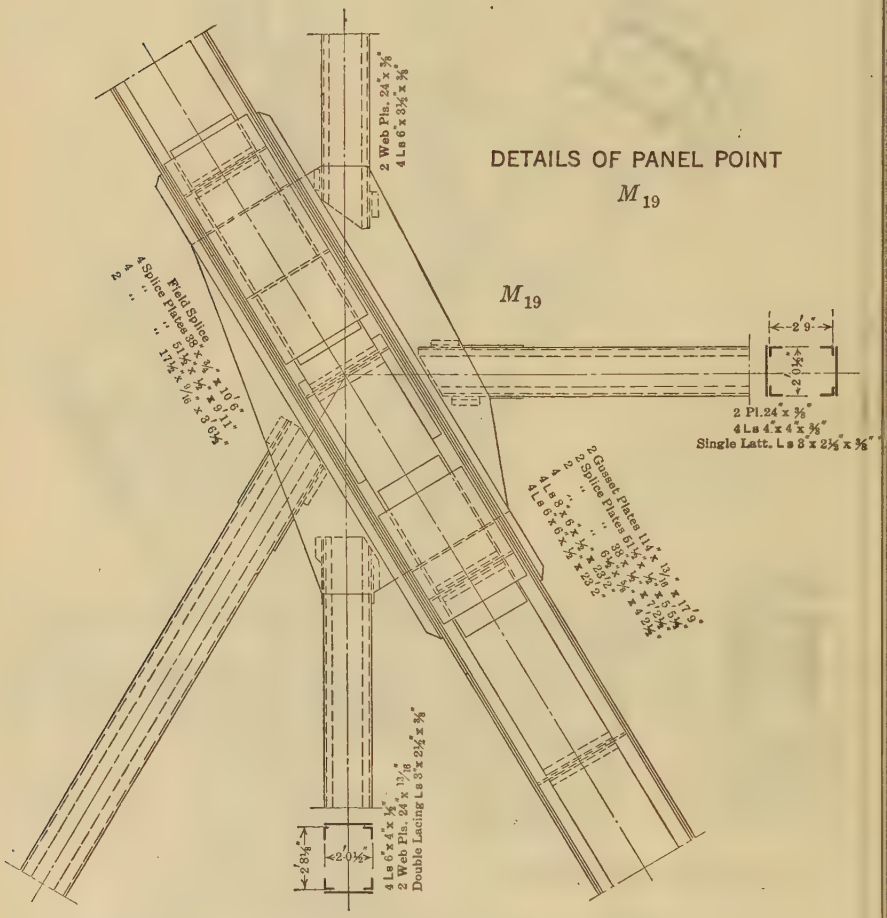

DETAILS OF PANEL POINT
M_{19}
M_{19}
2 Web Pls. 24" x ⅜"
4 Ls 6" x 3½" x ⅜"
Field Splice
4 Splice Plates 38" x ⅜" x 10' 6"
2 " " 51¾" x ⅜" x 9' 11"
" " 17¾" x ⅞" x 3' 0¾"
2'-2 9"
2 Pl. 24" x ⅜"
4 Ls 4" x 4" x ⅜"
Single Latt. Ls 3" x 2½" x ⅜"
2 Gusset Plates 114" x 13/8" x 17' 9"
2 " Splice Plates 51¾" x ⅜" x 5' 5¾"
4 Ls 8" x 6" x ⅜" x 6' 6"
4 Ls 8" x 6" x ½" x 23' 2"
2'-2 9"
2'-0½"
4 Ls 6" x 4" x ⅜"
2 Web Pls. 24" x 13/16"
Double Lacing Ls 3" x 2½" x ⅜"
2'-0½"

FIG. 7.

Bracing.—There is a lateral system along the bottom chords forming with the latter a two-span continuous truss. The laterals pass under and are connected to the bottom flange of the stringers.

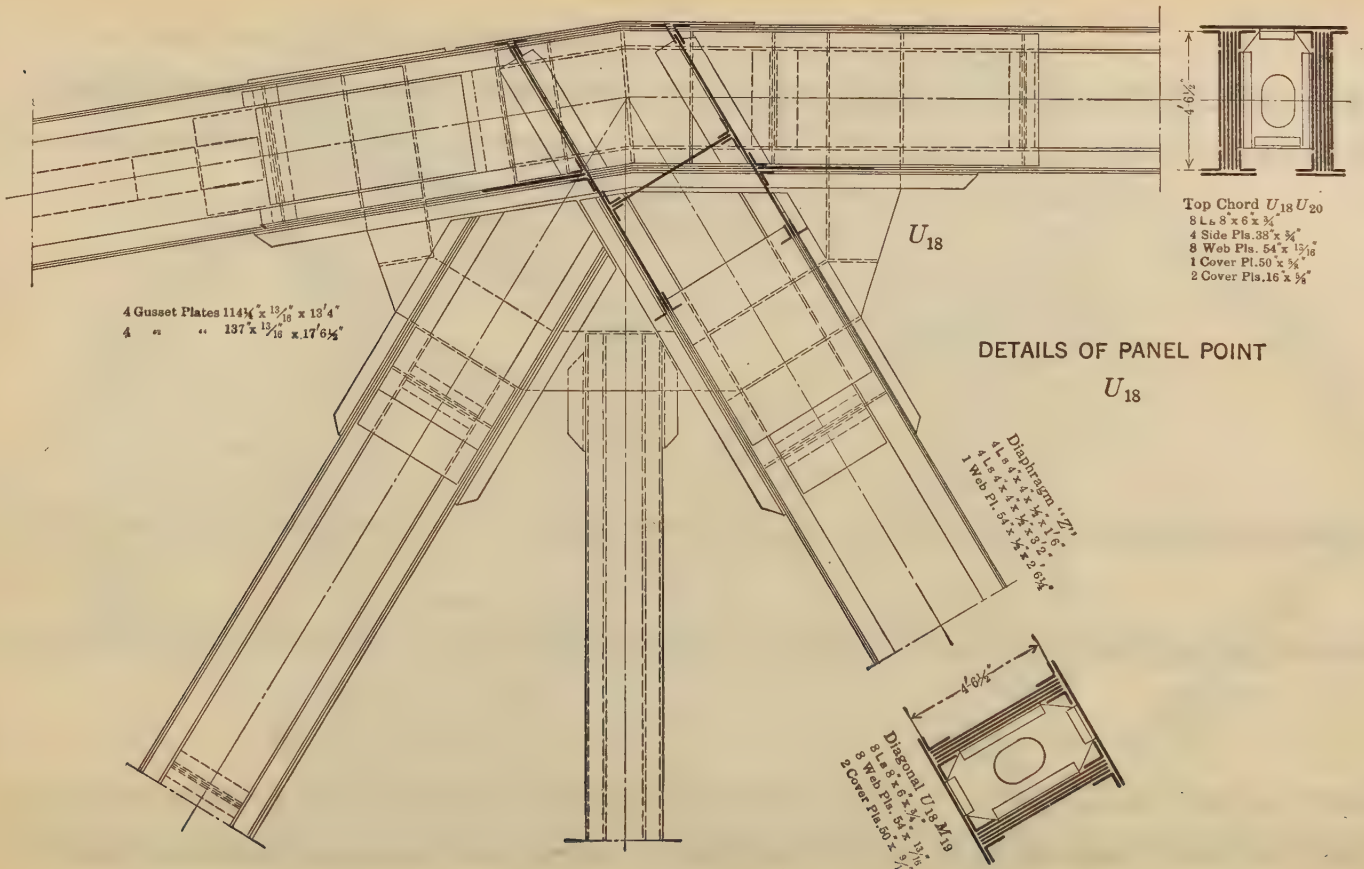

4 Gusset Plates 114¼" x 13/16" x 13'4"
4 " " 137" x 13/16" x 17'6½"
U_{18}
Top Chord $U_{18}U_{20}$
8 Ls 8" x 6" x ¾"
4 Side Pls. 38" x ¾"
8 Web Pls. 54" x 15/16"
1 Cover Pl. 50" x ⅝"
2 Cover Pls. 16" x ½"
DETAILS OF PANEL POINT
U_{18}
Diaphragm "Z"
4 Ls 4" x 4" x ½" x 1'6"
4 Ls 4" x 4" x ½" x 3'2"
1 Web Pl. 54" x ½" x 2'6¾"
Diagonal $U_{18}M_{19}$
8 Ls 5" x 5" x ¾"
8 Web Pls. 54" x 13/16"
2 Cover Pls. 50" x 13/16"

FIG. 8.

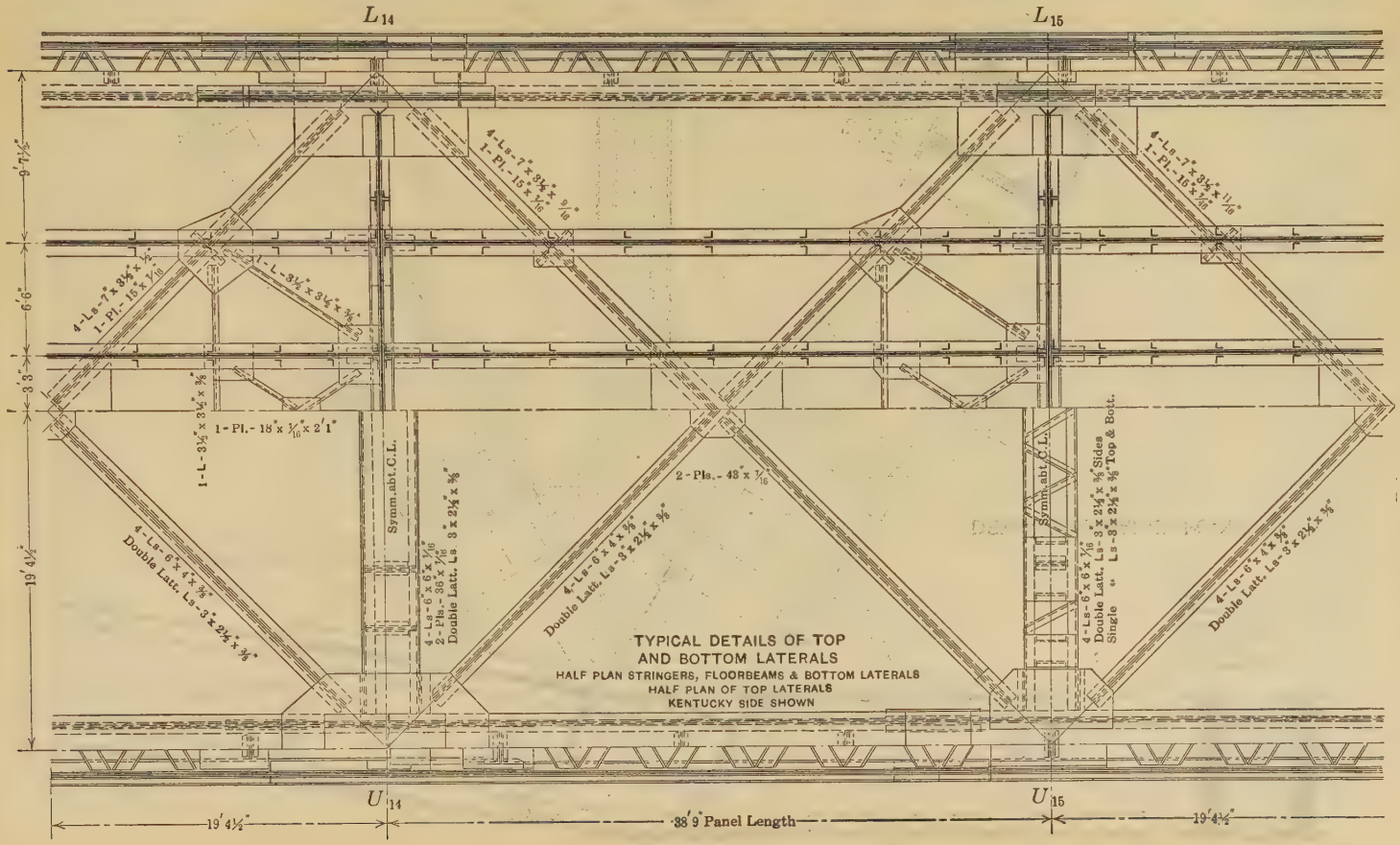

L_{14}
L_{15}
9'7½"
6'6"
3'3"
19'4½"
4-Ls-7"x3½"x½"
1-Pl.-15"x7/16"
4-Ls-7"x3½"x 9/16"
1-Pl.-15"x9/16"
4-Ls-7"x3½"x 11/16"
1-Pl.-15"x11/16"
1-L-3½"x3½"x⅜"
1-L-3½"x3½"x⅜"
1-Pl.-18"x1/16"x2'1"
Symm.abt.C.L.
4-Ls-6"x6"x7/16"
2-Pls.-36"x1/16"
Double Latt. Ls 3"x2½"x⅜"
2-Pls.-48"x1/16"
4-Ls-6"x4"x⅜"
Double Latt. Ls-3"x2½"x⅜"
4-Ls-6"x4"x⅜"
Double Latt. Ls-3"x2½"x⅜"
Symm.abt.C.L.
4-Ls-6"x6"x⅜"
Double Latt. Ls 3"x2½"x⅜"Sides
Single " Ls-3"x2½"x⅜"Top & Bott.
4-Ls-6"x4"x⅜"
Double Latt. Ls-3"x2½"x⅜"
TYPICAL DETAILS OF TOP
AND BOTTOM LATERALS
HALF PLAN STRINGERS, FLOORBEAMS & BOTTOM LATERALS
HALF PLAN OF TOP LATERALS
KENTUCKY SIDE SHOWN
U_{14}
U_{15}
19'4½"
38'9" Panel Length
19'4½"

FIG. 9

Fig. 10.—Box Girder Portals Between Diagonals Over Center Pier, Scioto ville Bridge.

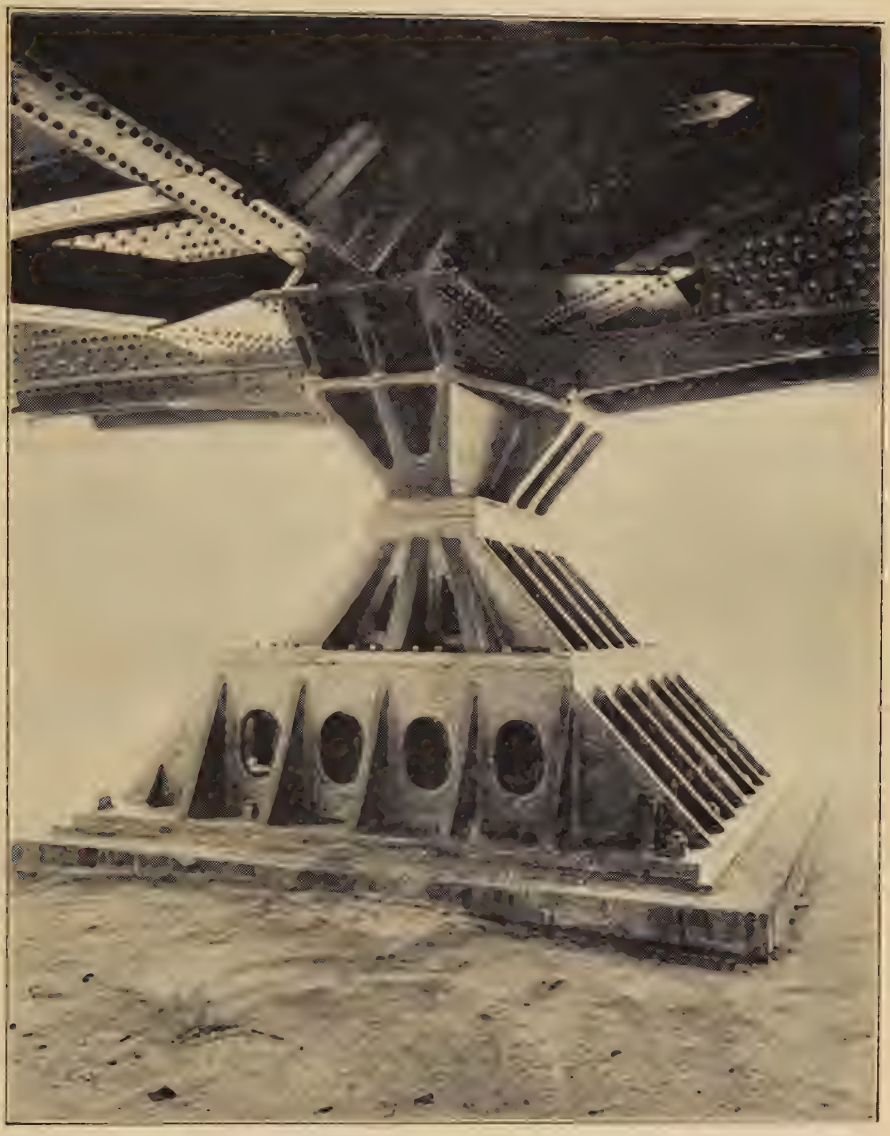


FIG. 11.—STEEL CASTING ON CENTER PIER, SCIOTOVILLE BRIDGE.

There is also a lateral system between the top chords from end to end. The portions between panel points, U_2 and U_8, are assumed to act as simple spans, transmitting the end shears at these points to the portals between the inclined posts. The laterals and struts of this system are of the full depth of the top chord (Fig. 9).

The portals between the end posts and between the inclined posts at the center pier are very rigid. The upper part consists of rigid single intersection braces and the lower one of a solid plate-girder arch (Fig. 10).

Sway-frames between the other web members have been purposely omitted as being unnecessary and as they would have had to be made very strong to resist unequal deflections of the trusses. Instead, there is a lattice frame at every panel point which acts as a lateral brace for the long web members and, at the same time, as a strut between the upper ends of the U-shaped floor-beam, the latter acting as an inverted arch, as described later.

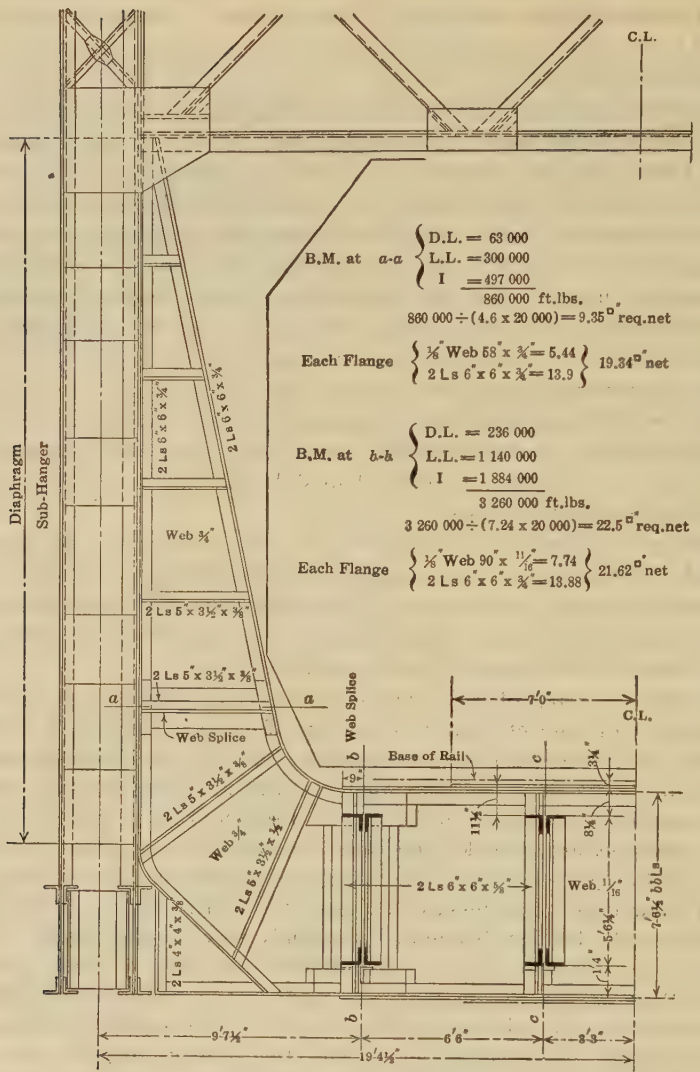
The longitudinal struts which brace the long verticals at mid-height, extend over two panels for better appearance. They were also useful in the cantilever erection of the Kentucky span. After erection, the connection at one end was loosened so as to allow it to slide and thus prevent the introduction of indeterminate stresses.

Bearings.—All bearings are of cast steel. The bearings at the center pier are fixed; those at the ends are movable. The longitudinal force from braking and traction is thus transmitted to the center pier which carries about 60% of the vertical load of the entire bridge, the longitudinal expansion of the bridge being divided between the two end bearings.

The center bearing of each truss carries a total vertical load of 16 406 000 lb., inclusive of impact, and it transmits, further, a longitudinal force of 2 520 000 lb. It consists of the shoe-casting (Fig. 11), bolted to the truss, and a pedestal made up of three individual castings arranged in two tiers. The truss transmits its reactions to the shoe in direct bearing by the gussets which, for this purpose, are extended somewhat below the bottom chord. The upper surface of the shoe-casting is planed slightly convex parallel to the truss to prevent dangerous edge pressure, although the casting has been proportioned by assuming the load as distributed uniformly over its full length.

To concentrate the reactions and permit the truss to deflect freely, the lower surface of the shoe-casting was planed to a cylindrical surface with a radius of 1 150 in., whereas the top surface of the pedestal was finished to a perfect plane. Under maximum load, the contact area is about 8½ in. wide, and the greatest pressure along the center line of this area is 29 900 lb. per sq. in., the average being 23 300 lb.

The reactions of the bottom lateral truss are transmitted from the lateral gussets through diaphragms to a lateral extension of the shoe-casting in order to relieve the gussets of the main trusses of transverse bending. The longitudinal force is transmitted from the main gussets to vertical lugs of the shoe-casting by means of turned bolts 2½ in. in diameter. To prevent horizontal motion, the shoe-casting is secured to the pedestal by four steel dowels 6 in. in diameter.


C.L.
B.M. at a-a
D.L. = 63 000
L.L. = 300 000
I = 497 000
860 000 ft.lbs.
860 000 ÷ (4.6 x 20 000) = 9.35 □" req.net
Each Flange
⅛" Web 58" x ¾" = 5.44
2 Ls 6" x 6" x ¾" = 13.9
19.34 □" net
B.M. at b-b
D.L. = 236 000
L.L. = 1 140 000
I = 1 884 000
3 260 000 ft.lbs.
3 260 000 ÷ (7.24 x 20 000) = 22.5 □" req.net
Each Flange
⅛" Web 90" x 11/16" = 7.74
2 Ls 6" x 6" x ¾" = 13.88
21.62 □" net
Diaphragm
Sub-Hanger
2 Ls 6" x 6" x ¾"
Web ¾"
2 Ls 5" x 3½" x ⅜"
Web Splice
Base of Rail
2 Ls 6" x 6" x ⅝"
Web 11/16"
9'7½"
6'6"
3'3"
19'4½"

INTERMEDIATE FLOORBEAM

Assumed Wind, Lateral & Braking Force

Wind along Top Chord = 800 lb. per lin.ft.of Bridge
" " Bottom Chord = 700 " " " " " "
" on Train. = 500 " " " " " "
Lateral Force from Train = 10% of Live Load (E.60) on one Track
Braking Force = 1000 lb. per lin.ft.of Bridge on one Track
or: $1000 \frac{25.875}{38.75}$ = 670 lb. per lin.ft.of one Truss

FIG. 12.

The complete bearing weighs 75 tons and the heaviest individual casting, 21 tons. The end bearings are much lighter and are of the ordinary type with a pin, 6 in. in diameter, and a nest of rockers, 16 in. high. Each end bearing weighs, complete, 20 tons.

Floor System.—The tracks are carried by four lines of stringers of the usual plate-girder type, which are framed into the floor-beams.

The floor-beams, however, are of exceptional design. They form U-shaped frames (Fig. 12), extending up to the bottom of the overhead struts. The available height of the floor was too shallow for an economical and stiff floor-beam of the ordinary type computed as a simple span between centers of trusses. There was, however, sufficient room available outside the train-clearance line for wide, deep brackets. By making the latter continuous with the floor-beam proper, it became admissible to compute the stresses by assuming the frame as an inverted two-hinged arch. This reduced the bending moments in the horizontal portion considerably and effected a substantial saving in weight. The overhead strut takes up the horizontal thrust of the inverted arch. The whole arrangement is contributive to stiffness and resistance to vibration, and the bridge behaves most satisfactorily in that regard under heavy trains.

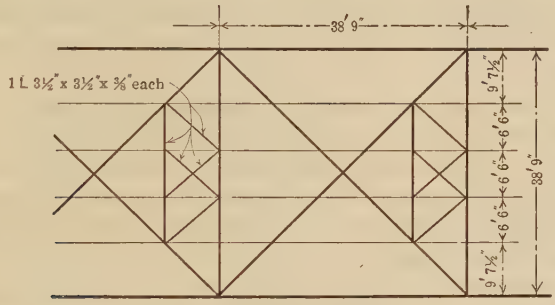


FIG. 13.

Braking Trusses.—In large bridges, traction or braking trusses are necessary to avoid excessive horizontal bending of the floor-beams by the longitudinal forces. In the Sciotoville Bridge, this has been solved in a novel way by providing such a truss in every panel in the plane of the bottom laterals (Fig. 13). In this manner, the stresses in the stringers from the longitudinal force are reduced to a negligible minimum and the horizontal bending of the floor-beams from that force and from the deformation (changes in length from tension and compression) of the bottom chords, is avoided.

No expansion joints are provided in the floor. This feature introduces high longitudinal stresses in the stringers from the deformation (extension and compression) of the bottom chords. To reduce these stresses to the

effect of the live load, the length of the stringers was made to correspond with the length of the bottom chords under full dead load.

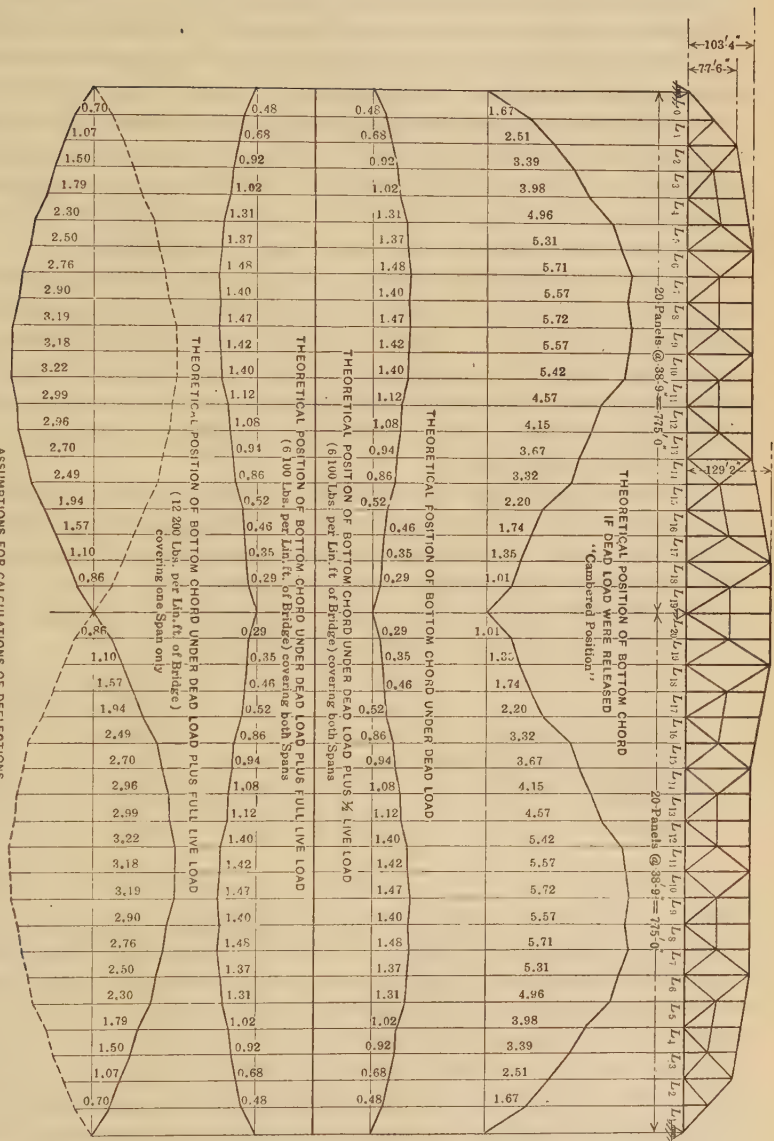
The stringer connections were not riveted until the spans had been swung. Even so, it was found necessary to increase the number of rivets in the stringer connections above that required for the vertical shear, by about 20%, or rather the diameter was increased from $\frac{7}{8}$ in. to 1 in. To avoid these high stresses in the stringer connections altogether, provision for an expansion joint in each panel was considered, but it was found that the additional expense for the expansion seats for the stringers would have been considerable and, moreover, the rigidity of the floor would have been greatly impaired.

An arrangement with only four expansion joints, eight panel lengths, or 310 ft., apart, with a braking truss in the middle between two joints, was also considered. Its advantage was a slight saving in weight of braking trusses and comparatively small stresses in the stringers and floor-beams, both from the longitudinal forces and the deformation of the bottom chords. In this case, however, it was not proper to connect the floor laterals rigidly to the stringers, because the deformation of the chords would be transmitted to the stringers through the laterals, and thus severely strain the latter and their connections to the stringers. To omit these connections would increase the weight and decrease the rigidity of the lateral system, which together with the undesirability of expansion joints, finally decided in favor of the adopted scheme.

Deflections, Camber, and Secondary Stresses.—The greatest live load deflections of the trusses are 3 in. from full load covering both spans and $4\frac{5}{8}$ in., or about 1 : 2 000 of the span length, from full load covering only one span (Fig. 14). This is about the same as the deflection of a corresponding simple span having a height between one-sixth and one-seventh of the span length.

As was expected, the deflection polygon showed a rather sharp upward kink at the middle support and indicated high secondary stresses in the vicinity of this support. This was corroborated by a calculation of these stresses, which was made both for full load on one span only and for full load on both spans.

Without provision for reducing them, the largest secondary stresses would be 13 300 lb. per sq. in. from dead load, and 8 100 lb. per sq. in. from live load, or a total of 21 400 lb. per sq. in. in the bottom chord next to the center bearing, and 5 500 lb. per sq. in. from dead load, and 3 300 lb. per sq. in. from live load, or a total of 8 800 lb. per sq. in. in the inclined posts at the center pier. In view of this, and on account of the rigid truss connections, it was considered advisable to reduce the secondary stresses as far as possible, not only near the center support, but throughout the truss, in the chords as well as in the web members. This was done by cambering the trusses for full dead load plus one-half the live load, covering both spans, but assembling and erecting them so that the angles between the members and the bevels of the joints would correspond to the geometric form of truss. In other words, under


103'4"
77'6"
129'2"
20 Panels @ 38'9" = 775'0"
THEORETICAL POSITION OF BOTTOM CHORD IF DEAD LOAD WERE RELEASED "Cambered Position"
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD PLUS ½ LIVE LOAD (6 100 Lbs. per Lin. ft. of Bridge) covering both Spans
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD PLUS FULL LIVE LOAD (6 100 Lbs. per Lin. ft. of Bridge) covering both Spans
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD PLUS FULL LIVE LOAD (12 200 Lbs. per Lin. ft. of Bridge) covering one Span only
L0 L1 L2 L3 L4 L5 L6 L7 L8 L9 L10 L11 L12 L13 L14 L15 L16 L17 L18 L19 L20
0.70 1.07 1.50 1.79 2.30 2.50 2.76 2.90 3.19 3.18 3.22 2.99 2.96 2.70 2.49 1.94 1.57 1.10 0.86
0.48 0.68 0.92 1.02 1.31 1.37 1.48 1.40 1.47 1.42 1.40 1.12 1.08 0.94 0.86 0.52 0.46 0.35 0.29
0.48 0.68 0.92 1.02 1.31 1.37 1.48 1.40 1.47 1.42 1.40 1.12 1.08 0.94 0.86 0.52 0.46 0.35 0.29
1.67 2.51 3.39 3.98 4.96 5.31 5.71 5.57 5.72 5.57 5.42 4.57 4.15 3.67 3.32 2.20 1.74 1.35 1.01
1.01 1.35 1.74 2.20 3.32 3.67 4.15 4.57 5.42 5.57 5.72 5.57 5.71 5.31 4.96 3.98 3.39 2.51 1.67

DEFLECTION DIAGRAM

ASSUMPTIONS FOR CALCULATIONS OF DEFLECTIONS

Cross Sections of Members plus 25% Allowance for details
E = 30,000,000 lbs. per sq. in.
Dead Load — Unit Stresses as given on Stress Sheet
Live Load — Uniform Load equivalent to Cooper's E-60 or 12 200 lbs. per lin. ft. of Bridge
Trusses Cambered for Dead Load plus ½ Live Load Covering both Spans
Deflections given in inches

FIG. 14.

the load $\left(d + \frac{1}{2}\ l\right)$, the trusses are calculated to assume their true geometric form and the members to become straight and free of secondary stresses.

The secondary stresses from dead load are thus eliminated and those from live load are halved or, more properly expressed, the secondary stresses under dead load only are equal, but of opposite sign, to those under full live load covering both spans, and, in absolute value, equal to one-half of those which would be produced by full live load if the angles between the members would correspond to the cambered form of truss. The largest secondary stresses, previously mentioned, are thus reduced to 4 000 lb. per sq. in. in the bottom chord and to 1 650 lb. per sq. in. in the inclined posts, which stresses are fully covered by the margin of safety of the direct stresses.

In order to secure the greatest safety during erection, and because no reliance was placed on the turning of the ends of the truss members as the erection proceeded, it was decided to follow the erection of the members with the riveting as soon as practicable and before the members would take any appreciable stress. This meant initial bending of all truss members for making their connections, which was accomplished partly by drifting and partly by special jacking operations.

This is the first bridge, in which this method of reducing secondary stresses in all members has been used. It is a great improvement on previous practice, in which secondary stresses were assumed to take care of themselves, at the risk of overstressing the metal at the connections.

Steel Weights.—The entire bridge of two continuous spans, each 775 ft. long, contains 13 240 tons of steelwork, as follows:

	Total weight, in pounds.	Weight per 1 ft. of bridge, in pounds.
Floor system	3 403 200	2 200
Floor lateral bracing	732 400	475
Overhead bracing	1 942 400	1 250
Trusses	19 947 400	12 880
Bearings	456 200	295
Total steelwork	26 481 600	17 100

The weight of the metal in the two riveted simple girder spans, each 775 ft. long, of the usual type, and for the same live load, would have been 16 000 tons, or 20% heavier than the design herein described and as built, not counting considerable extra metal which would have been required for the erection of the channel span by the cantilever method.

As the proportion of dead load to live load in these continuous spans is as 3:2, an increase of live load from E-60 to E-75 would increase the total stresses from dead plus live load on the average only 10 per cent. That leaves a large margin of safety for the heavier loading if it is ever used.

The truss span nearest in length is the 720-ft., double-track, simple girder span of the Ohio River Bridge at Metropolis. The metal weighs 8 023 500 lb., or 11 200 lb. per lin. ft. of bridge, the light weight being due to alloy steel, eye-bars instead of riveted tension members, and the use of higher unit stresses.

5.—Fabrication and Erection of Steelwork

Quality of Steel.—All parts, except the rivets and the cast-steel bearings, are of open-hearth structural steel. The chemical and physical requirements for the various grades are as given in Table 1.

TABLE 1.

	Structural steel.	Rivet steel.	Cast steel.
Phosphorus, maximum basic	0.04	0.04	0.05
" " acid	0.06	0.04	0.08
Sulphur, maximum	0.05	0.04	0.05
Ultimate strength, in pounds per square inch, maximum	70 000	58 000	.....
Ultimate strength, in pounds per square inch, desired	66 000	.....	.....
Ultimate strength, in pounds per square inch, minimum	62 000	50 000	65 000
Yield point, minimum	35 000	28 000	33 000
Elongation, minimum in 2 in. for cast steel; in 8 in. for other steel	22%	28%	20%
Character of fracture	Silky	Fine silky	Silky or fine granular.
Cold bend without fracture	180° around pin of thickness of test piece.	180° flat	90° around pin three times thickness of test piece.

Workmanship.—The principal requirements for workmanship were as follows: Punching was allowed to full size of the hole in the material up to $\frac{1}{2}$ in. thick; to $\frac{1}{16}$ in. smaller than the finished hole in material up to $\frac{5}{8}$ in. thick; to $\frac{1}{8}$ in. in diameter for rivets $\frac{7}{8}$ in. and more, in material $\frac{1}{8}$ in. and $\frac{3}{4}$ in. thick; and to $\frac{3}{4}$ in. in diameter for rivets 1 in. and more, in material $\frac{1}{8}$ in. thick.

All holes punched small had to be reamed or drilled to full size after the assembling of parts. All other holes had to be either drilled from solid to full size after assembling, or drilled small and reamed to full size after assembling. As most of the material is $\frac{1}{8}$ in. thick, or less, these specifications provided an opportunity to punch holes to the widest possible extent and yet insure the removal of all material possibly injured by the punching. All sheared edges of material more than $\frac{1}{2}$ in. in thickness had to be planed off at least one-quarter of the thickness.

To insure perfect filling of holes, rivets with a grip less than four times their diameter had to have, when cold, a diameter not less than $\frac{3}{64}$ in. smaller than the finished hole where the holes were reamed or drilled after assembling, and not less than $\frac{1}{8}$ in. smaller than the finished hole where the holes were punched full size (the latter occurred generally only in less important members). Rivets of a grip of four or more times the diameter had to be tapered to the same size and shape as those for the Hell Gate Bridge.*

Shop Assembling of Trusses.—The specifications prescribed that the connecting parts of the riveted trusses should be accurately laid out and assembled at the shop and that all rivet holes should be reamed or drilled in that position.

The complete assembling of the trusses or, at least, of a series of complete panels, would have assured the greatest accuracy and least chance for errors. This, however, was impracticable because, as previously explained, the angles

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 920.

between the members had to conform to the "geometric" form of the truss, whereas the length of the members in their unstrained condition conformed to the "cambered" form of truss.

To assemble any panel or group of panels completely would have required forcible bending of the members and drilling of the holes in that condition, a very difficult and expensive operation. The trusses, therefore, were assembled in sections, as shown on Fig. 15, by connecting the web members to each chord separately. The members were carefully leveled and laid out with a transit to the correct angles, and the distances were carefully measured with a steel tape. The measurements were usually made in the early morning when the temperature was uniform in all parts.

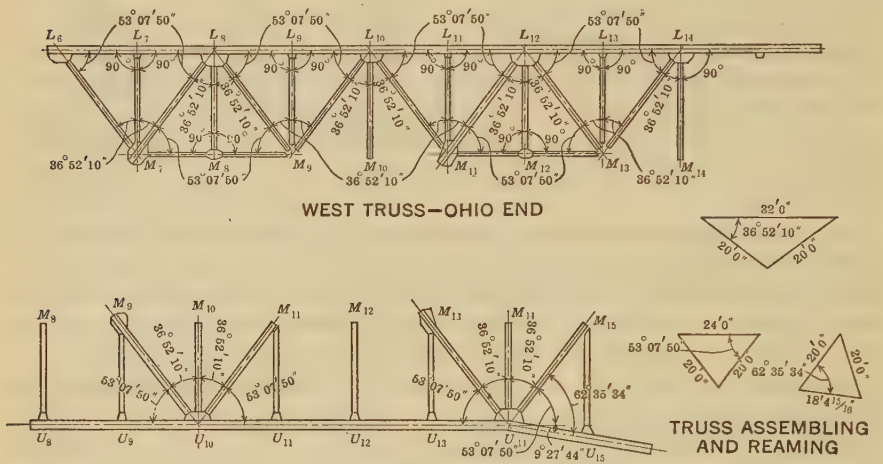


FIG. 15.

Connections were made with a sufficient number of $\frac{5}{8}$-in. tack-bolts to hold the members firmly together during the reaming and drilling of the holes. The holes for these bolts had been previously punched to $1\frac{1}{8}$ in. in diameter, and all other holes were then drilled from the solid. For this purpose, all the holes in the top plate of each web, in the position in which the members were laid for assembling, had been previously punched to $1\frac{1}{8}$ in., and this plate served as a template for drilling the holes. Finally, the holes which served for the tack-bolts, were reamed to full size.

The chords were laid first to an exact straight line, the joints being brought to perfect and forcible contact, and the reaming and drilling of the holes of the splices was started before the web members were assembled. The two halves of the main diagonals, having a splice at the *M*-points, were assembled to each other in the shop to a straight line, then taken apart and re-assembled to the chords in the assembling yard.

The sub-diagonals, hangers, and horizontal struts were first connected up and reamed at one end, then shifted slightly for connecting up and reaming of the other end. While one end of the sub-diagonals was being reamed, the

other end was held in position by a temporary connection through a few extra holes provided for that purpose, which, later, were plugged. Connections of sub-posts, which are of secondary importance, were allowed to be reamed to an iron templet. All connections were match-marked before the assembled sections were dismantled.

Erection.—A detailed description of the erection of the steel superstructure which offered many difficult and unprecedented problems, has been given in a series of articles* by Clyde B. Pyle, M. Am. Soc. C. E., Field Engineer for the McClintic-Marshall Company, the contractors for the fabrication and erection of the steelwork. Therefore, only the general procedure and the conditions and considerations which governed the method of erection, will be mentioned here.

The War Department required the maintenance, during erection, of a minimum clear opening of 420 ft. over the river channel on the Kentucky side, for navigation. It was evident, therefore, that this part of the Kentucky span had to be erected by the cantilever method.

No opening for navigation was required under the Ohio span and this span lent itself better for falsework erection also on account of the shallow bottom. Even with that span, however, great risks had to be taken because of the danger of the falsework being carried away by a sudden flood, especially as no piles could be driven on account of the rock bottom. To minimize this danger the contractor used narrow falsework towers placed under the main panel points only, instead of closely spaced bents, thus leaving openings of about 60 ft. under the main portion of the span for the passage of drift and ice.

The erection of the entire Kentucky span as a cantilever, without intermediate support, would have required heavy additions to the chords and some web members near the middle pier. The plans, therefore, contemplated the erection of the four end panels near the Kentucky shore on falsework, with two panels cantilevering beyond, and the other fourteen panels by cantilevering from the center pier. This scheme would have required only a comparatively small addition to some of the members. The six panels on the Kentucky side would have been erected simultaneously with the other portion, which would have saved considerable time. It would have required, however, a separate erection plant on the Kentucky side, and, moreover, the connection of the two joining arms would have presented difficulties. For these reasons, the contractor preferred to erect the entire span as cantilever from the center pier, giving, however, intermediate support by steel bents at the eighth and fourth panel points from the end pier, as soon as these points were reached. (Plate XII.) Thus, the free cantilever portion was reduced to twelve panels, or 465 ft. Even so, it was the longest cantilever truss ever erected. The two falsework towers near the center pier on the Kentucky side were placed merely for convenience in erecting the extremely heavy panels near that pier by means of the gantry traveler, but did not assist in the cantilever erection of the remainder of the Kentucky span.

One of the principal features of the erection of this bridge was the initial bending of the members, which was necessary in order to reduce the secondary

* *Engineering News-Record*, January 10th and 31st and December 26th, 1918.

stresses, as explained previously. These operations, as well as the adjustments in height of the trusses at the end piers and the temporary intermediate supports, required elaborate preparations and special jacking devices (Fig. 16) as fully described in the article by Mr. Pyle. The contractors deserve full credit for the careful and elaborate manner in which the operations were prepared and successfully executed.

In the main, the erection procedure was as follows: By means of a gantry traveler, the falsework and on it the steel floor system and delivery tracks were laid from the pier on the Ohio shore to two panels beyond the center pier on the Kentucky side. (Fig. 17.) Then, working toward the pier on the Ohio shore, the traveler laid the bottom chords which were riveted at once while lying in a straight line and were then jacked to the desired camber. In this position, the Ohio end was 8¼ in. lower than its final position.

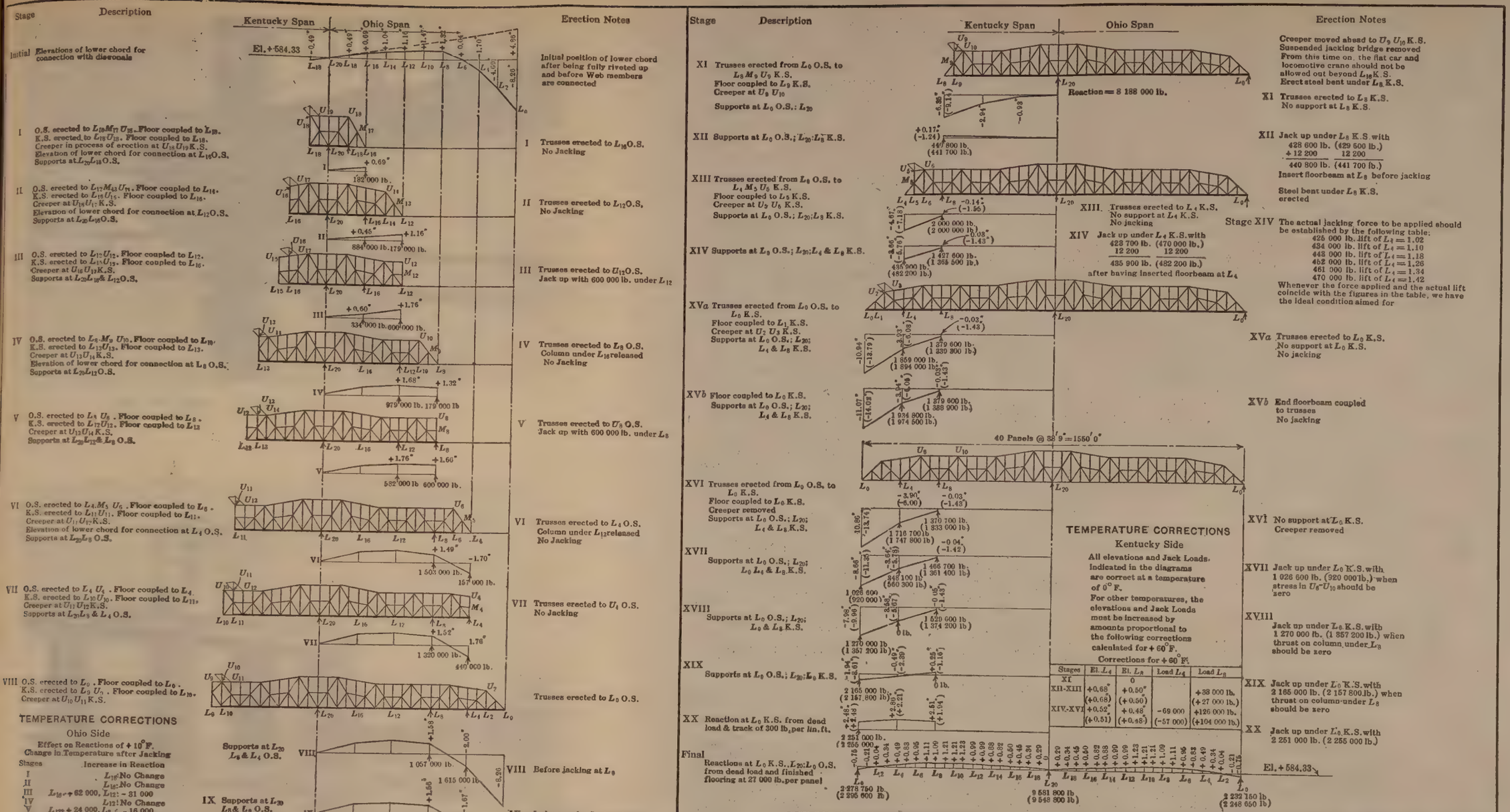
The traveler in the meantime having been raised to its full height and brought back to the center pier, the erection of the trusses was then proceeded with, working toward the Ohio end. (Fig. 18.) It had been intended originally to proceed simultaneously with the cantilever erection of the Kentucky span, but, on account of a shortage of labor and the approaching winter and high-water season, with its dangers to the falsework, all efforts were concentrated on the Ohio span in order to hasten its completion.

Consequently, the creeper traveler which, in the meantime, had been placed on top of the trusses over the center pier, had erected only one panel on the Kentucky side by the time the Ohio span was completely connected up (Fig. 19). The creeper traveler then proceeded with the cantilever erection of the Kentucky span and, at the same time, the timber falsework under the Ohio span was gradually removed, leaving only the steel columns under Panel Points 4, 8, 12, and 16 to support the trusses (Fig. 20). The releasing of these columns was finally accomplished by jacking the Ohio end of the span to its final position, when the Kentucky cantilever had reached about mid-span. The jacking was done by one 500-ton and four 200-ton hydraulic jacks under each truss (Fig. 21).

The Kentucky cantilever, having reached the eighth panel point from the end (Fig. 20), was jacked up 7⅜ in. from the steel bent erected at that point. This procedure was repeated when the truss reached the next bent at the fourth panel point from the end. The jacking height at that point was 1 in. (Fig. 22). When the truss reached the pier on the Kentucky shore (Fig. 23), it had a deflection of 16¼ in. It was then jacked up to its final position and placed on the rocker bearings, whereby the intermediate supports were released of their load. The final jacking force agreed so closely with calculated reactions that no further adjustment was necessary. Fig. 24 shows a pair of the top chord members being lifted simultaneously.

The operations described indicate sufficiently the sensitiveness of the structure from variations of deflections during erection and the necessity for their accurate analysis and computation in advance in order to insure an exact fit in the connections.

The erection of the steel work was started in June, 1916, and the bridge was completed in August, 1917. From the beginning of the work on the coffer-


Stage
Description
Kentucky Span
Ohio Span
Erection Notes
Initial
Elevations of lower chord for connection with diseonals
El. + 584.33
Initial position of lower chord after being fully riveted up and before Web members are connected
I
O.S. erected to $L_{18}M_{17}U_{18}$. Floor coupled to L_{18}. K.S. erected to $L_{18}U_{19}$. Floor coupled to L_{18}. Creeper in process of erection at $U_{18}U_{19}$ K.S. Elevation of lower chord for connection at L_{16} O.S. Supports at $L_{20}L_{18}$ O.S.
I Trusses erected to L_{16} O.S. No Jacking
II
O.S. erected to $L_{12}M_{13}U_{14}$. Floor coupled to L_{16}. K.S. erected to $L_{16}U_{15}$. Floor coupled to L_{16}. Creeper at $U_{16}U_{17}$ K.S. Elevation of lower chord for connection at L_{12} O.S. Supports at $L_{20}L_{16}$ O.S.
II Trusses erected to L_{12} O.S, No Jacking
III
O.S. erected to $L_{12}U_{12}$. Floor coupled to L_{12}. K.S. erected to $L_{15}U_{15}$. Floor coupled to L_{16}. Creeper at $U_{16}U_{17}$ K.S. Supports at $L_{20}L_{16}$ & L_{12} O.S.
III Trusses erected to U_{12} O.S. Jack up with 600 000 lb. under L_{12}
IV
O.S. erected to $L_8M_9U_{10}$. Floor coupled to L_{10}. K.S. erected to $L_{13}U_{13}$. Floor coupled to L_{13}. Creeper at $U_{13}U_{14}$ K.S. Elevation of lower chord for connection at L_8 O.S. Supports at $L_{20}L_{12}$ O.S.
IV Trusses erected to L_8 O.S. Column under L_{10} released No Jacking
V
O.S. erected to L_8U_8. Floor coupled to L_8. K.S. erected to $L_{12}U_{12}$. Floor coupled to L_{12} Creeper at $U_{13}U_{14}$ K.S. Supports at $L_{20}L_{12}$ & L_8 O.S.
V Trusses erected to U_8 O.S. Jack up with 600 000 lb. under L_8
VI
O.S. erected to $L_4M_5U_6$. Floor coupled to L_6. K.S. erected to $L_{11}U_{11}$. Floor coupled to L_{11}. Creeper at $U_{11}U_{12}$ K.S. Elevation of lower chord for connection at L_4 O.S. Supports at $L_{20}L_8$ O.S.
VI Trusses erected to L_4 O.S. Column under L_{12} released No Jacking
VII
O.S. erected to L_4U_4. Floor coupled to L_4. K.S. erected to $L_{10}U_{10}$. Floor coupled to L_{11}. Creeper at $U_{11}U_{12}$ K.S. Supports at $L_{20}L_8$ & L_4 O.S.
VII Trusses erected to U_4 O.S. No Jacking
VIII
O.S. erected to L_0. Floor coupled to L_0. K.S. erected to L_9U_9. Floor coupled to L_{10}. Creeper at $U_{10}U_{11}$ K.S.
Trusses erected to L_0 O.S.
Supports at L_{20} L_8 & L_4 O.S.
VIII Before jacking at L_8
IX Supports at L_{20} L_8 & L_0 O.S. K.S. Same as for Stage VIII
IX Jack up under L_0 to release column under L_4
It is however not necessary to stop jacking at this intermediate stage
X Supports at L_{20} L_0 O.S. K.S. Same as for Stage VIII
X Continue jacking under L_0 to release column under L_8 and remove that column to L_4 K.S. Insert rollers under L_0 O.S. and bring L_0 O.S. to final position
182 000 lb.
884 000 lb. 179 000 lb.
334 000 lb. 600 000 lb.
979 000 lb. 179 000 lb
582 000 lb 600 000 lb.
1 503 000 lb. 157 000 lb.
1 320 000 lb. 440 000 lb.
1 057 000 lb. 1 615 000 lb.
1 145 000 lb. 0 lb. 1 238 000 lb.
1 926 000 lb.

TEMPERATURE CORRECTIONS

Ohio Side

Effect on Reactions of + 10° F. Change in Temperature after Jacking

Stages	Increase in Reaction
I	L_{16}: No Change
II	L_{16}: No Change
III	L_{16}: + 62 000, L_{12}: - 31 000
IV	L_{12}: No Change
V	L_{12}: + 24 000, L_8: - 16 000
VI	L_8: No Change
VII	L_8: + 13 000, L_4: - 10 000
VIII	L_8: + 13 000, L_4: - 10 000
IX	L_0: L_8:
X	L_0: No Change

Each steel column (under L_4L_8, L_{12} and L_{16}) expands ½ inch for a temperature rise of 60° F. The temperatures should therefor be noted and reported whenever levels are taken and whenever jacking is done

Stage
Description
Kentucky Span
Ohio Span
Erection Notes
XI Trusses erected from L_0 O.S. to $L_8M_9U_9$ K.S. Floor coupled to L_9 K.S. Creeper at U_9U_{10} Supports at L_0 O.S.; L_{20}
Reaction = 8 188 000 lb.
XI Trusses erected to L_8 K.S. No support at L_8 K.S.
XII Supports at L_0 O.S.; L_{20}; L_8 K.S.
440 800 lb. (441 700 lb.)
XII Jack up under L_8 K.S. with 428 600 lb. (429 600 lb.) + 12 200 12 200 = 440 800 lb. (441 700 lb.) Insert floorbeam at L_8 before jacking
Steel bent under L_8 K.S. erected
XIII Trusses erected from L_0 O.S. to $L_4M_5U_5$ K.S. Floor coupled to L_5 K.S. Creeper at U_5U_6 K.S. Supports at L_0 O.S.; L_{20}; L_8 K.S.
2 000 000 lb. (2 000 000 lb.)
XIII Trusses erected to L_4 K.S. No support at L_4 K.S. No jacking
XIV Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
1 427 600 lb. (1 365 500 lb.)
435 900 lb. (482 200 lb.)
XIV Jack up under L_4 K.S. with 423 700 lb. (470 000 lb.) 12 200 12 200 = 435 900 lb. (482 200 lb.) after having inserted floorbeam at L_4
Stage XIV The actual jacking force to be applied should be established by the following table:
425 000 lb. lift of L_8 = 1.02
434 000 lb. lift of L_4 = 1.10
443 000 lb. lift of L_4 = 1.18
452 000 lb. lift of L_4 = 1.26
461 000 lb. lift of L_4 = 1.34
470 000 lb. lift of L_4 = 1.42
Whenever the force applied and the actual lift coincide with the figures in the table, we have the ideal condition aimed for
XVa Trusses erected from L_0 O.S. to L_0 K.S. Floor coupled to L_1 K.S. Creeper at U_2 U_3 K.S. Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
1 879 800 lb. (1 329 800 lb.)
1 859 000 lb. (1 894 000 lb.)
XVa Trusses erected to L_0 K.S. No support at L_0 K.S. No jacking
XVb Floor coupled to L_0 K.S. Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
1 879 600 lb. (1 389 900 lb.)
1 934 800 lb. (1 974 500 lb.)
XVb End floorbeam coupled to trusses No jacking
40 Panels @ 38′ 9″ = 1550′ 0″
XVI Trusses erected from L_0 O.S. to L_0 K.S. Floor coupled to L_0 K.S. Creeper removed Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
1 370 700 lb. (1 833 000 lb.)
1 716 700 lb. (1 747 800 lb.)
XVI No support at L_0 K.S. Creeper removed
XVII Supports at L_0 O.S.; L_{20}; L_0 L_4 & L_8 K.S.
1 466 700 lb. (1 361 400 lb.)
846 100 lb. (560 300 lb.)
1 026 600 (920 000 lb.)
XVII Jack up under L_0 K.S. with 1 026 600 lb. (920 000 lb.) when stress in U_6–U_{10} should be zero
XVIII Supports at L_0 O.S.; L_{20}; L_0 & L_8 K.S.
1 520 600 lb (1 374 200 lb.)
0 lb.
1 270 000 lb (1 357 200 lb.)
XVIII Jack up under L_0 K.S. with 1 270 000 lb. (1 357 200 lb.) when thrust on column under L_8 should be zero
XIX Supports at L_0 O.S.; L_{20}; L_0 K.S.
2 165 000 lb. (2 157 800 lb.)
2 251 000 lb. (2 255 000 lb.)
XIX Jack up under L_0 K.S. with 2 165 000 lb. (2 157 800 lb.) when thrust on column under L_8 should be zero
XX Reaction at L_0 K.S. from dead load & track of 300 lb. per lin. ft.
XX Jack up under L_0 K.S. with 2 251 000 lb. (2 255 000 lb.)
Final Reactions at L_0 K.S., L_{20}; L_0 O.S. from dead load and finished flooring at 27 000 lb. per panel
El. + 584.33
2 278 750 lb. (2 295 600 lb.)
9 581 900 lb. (9 548 800 lb.)
2 232 150 lb. (2 248 650 lb.)

TEMPERATURE CORRECTIONS

Kentucky Side

All elevations and Jack Loads, indicated in the diagrams are correct at a temperature of 0° F.

For other temperatures, the elevations and Jack Loads must be increased by amounts proportional to the following corrections calculated for + 60° F.

Corrections for + 60° F.

Stages	El. L_4	El. L_8	Load L_4	Load L_8
XI		0		
XII-XIII	+ 0.66″ (+ 0.68)	+ 0.50″ (+ 0.50)		+ 38 000 lb. (+ 27 000 lb.)
XIV-XVI	+ 0.52″ (+ 0.51)	+ 0.48″ (+ 0.48)	- 69 000 (- 57 000)	+ 126 000 lb. (+ 104 000 lb.)

DEFLECTION CONSTANTS (Ky. Side)

	Deflection at L_0	Deflection at L_4	Deflection at L_8
Bridge erected to L_9, Traveler at U_9 U_{10}			- 13.03 (- 15.82)
Bridge erected to L_4, Traveler at U_5 U_6		- 51.99 (- 63.10)	- 37.22 (- 45.03)
Bridge erected to L_0, Traveler at U_2 U_3	- 121.86 (- 148.06)	- 94.93 (- 115.12)	- 66.27 (- 80.20)
100 000 lb. at L_0	5.17 (6.27)	3.90 (4.72)	2.63 (3.19)
100 000 lb. at L_4	3.90 (4.72)	3.05 (3.69)	2.08 (2.51)
100 000 lb. at L_8	2.63 (3.19)	2.06 (2.51)	1.52 (1.84)
Column Compression for a load of 100 000 lb.	0	0 0193	0.0193

DIAGRAMS SHOWING PRINCIPAL ERECTION STAGES. DEFLECTIONS AND JACKING OPERATIONS

NOTES:- All Reactions and Jacking Loads are given in Pounds for one Truss. All Deflections are given in inches. Figures in parentheses are computed on the basis of no allowance made for details as affecting the cross-sections of the members. All other figures are computed on the basis of an addition to the cross-sections equivalent to 75% of the weight of the details of the members. Figures of elevations denote the height above or below a horizontal line through the L_{20} point. (El. Base of Rail + 590.08.) O. S. denotes Ohio Span. K. S. denotes Kentucky Span

Compare Mc Clintic Marshall Co.'s Drawings: D_1, D_2, D_3 & D_4

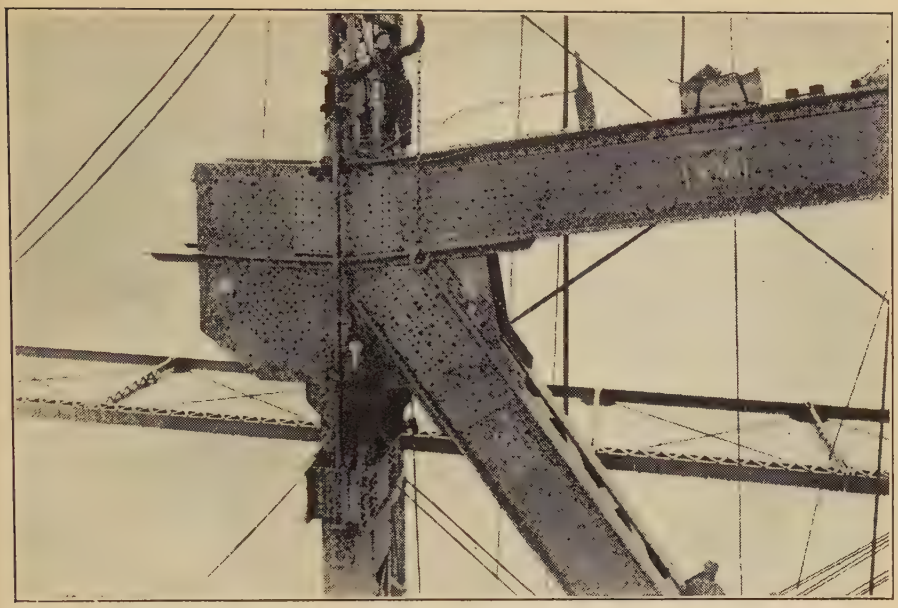


FIG. 16.—SPECIAL JACKING APPARATUS, SCIOTOVILLE BRIDGE.

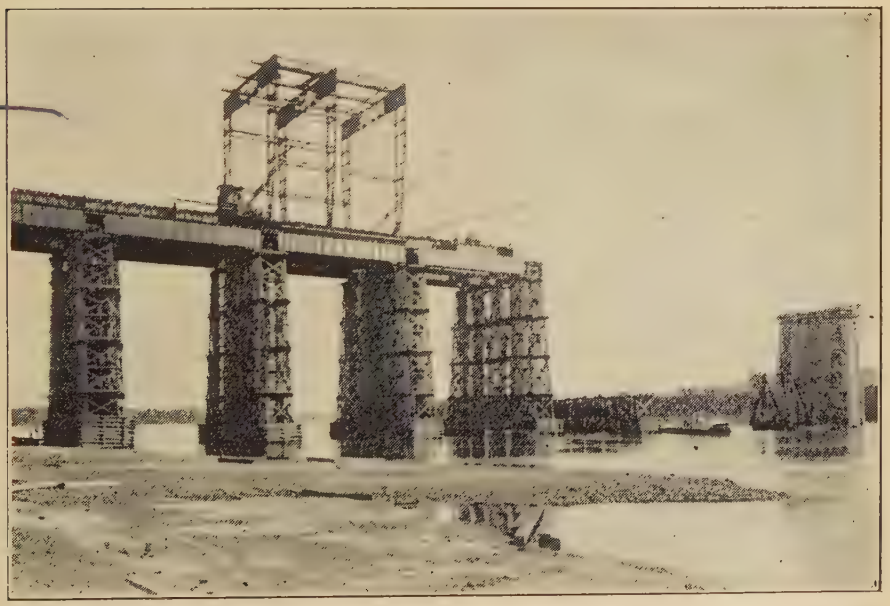


FIG. 17.—ERECTION OF FALSEWORK AND LAYING OF STEEL FLOOR SYSTEM AND TRACKS BY GANTRY TRAVELER, SCIOTOVILLE BRIDGE.

FIG. 18.—ERECTION OF TRUSSES, OHIO SPAN, SCIOTOVILLE BRIDGE.

FIG. 19.—ERECTION OF OHIO SPAN COMPLETED, SCIOTOVILLE BRIDGE.

FIG. 20.—CANTILEVER ERECTION, KENTUCKY SPAN, AND FALSEWORK UNDER OHIO SPAN REMOVED, SCIOTOVILLE BRIDGE.

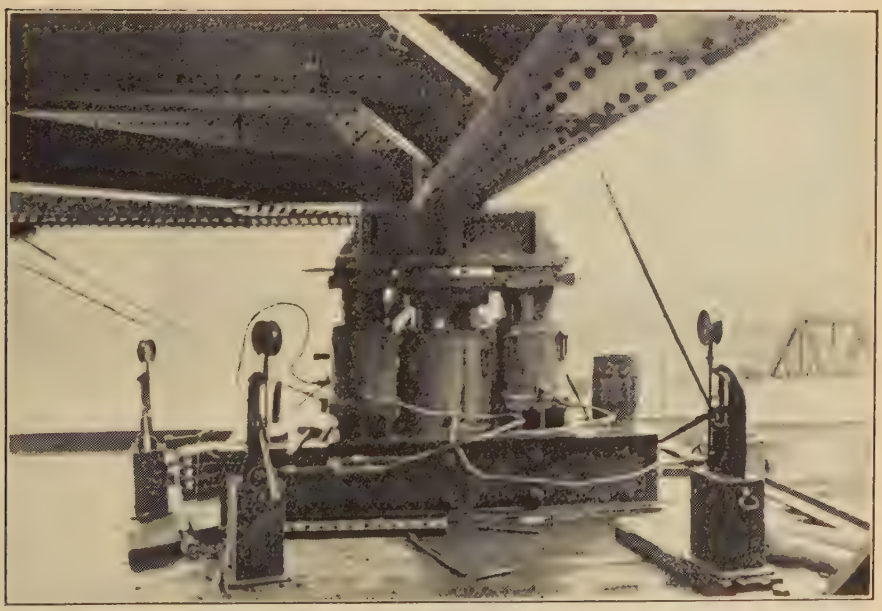


FIG. 21.—ADJUSTMENT OF KENTUCKY SPAN, SCIOTOVILLE BRIDGE, BY HYDRAULIC JACKS.

FIG. 22.—CANTILEVER SECTION, KENTUCKY SPAN, SCIOTOVILLE BRIDGE.

FIG. 23.—KENTUCKY SPAN, SCIOTOVILLE BRIDGE: TEMPORARY SUPPORTS READY FOR DISMANTLING.

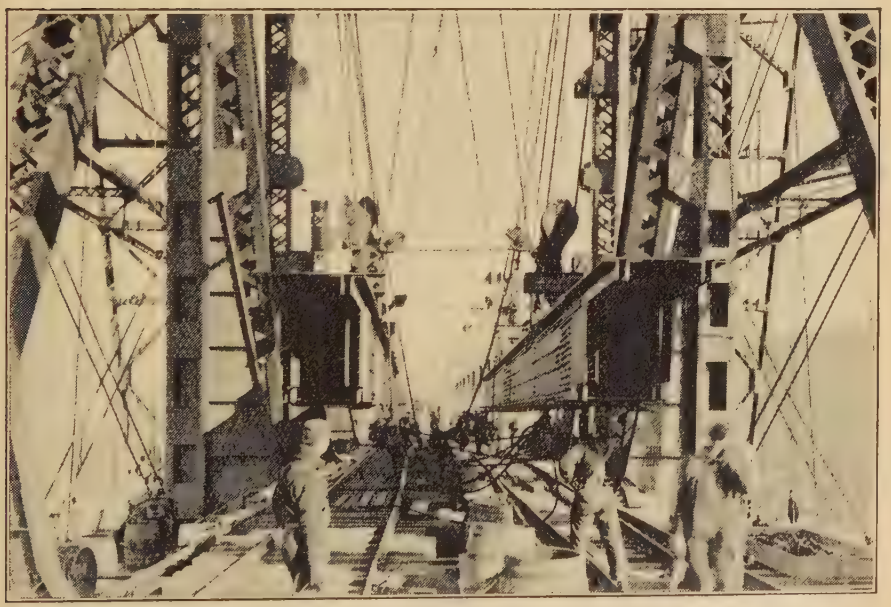


FIG. 24.—PAIR OF TOP CHORD MEMBERS, SCIOTOVILLE BRIDGE, BEING LIFTED SIMULTANEOUSLY.

dam for the middle pier in November, 1914, to the completion of the superstructure in August, 1917, in all 2 years and 10 months were required for the work which, under normal labor conditions, would have taken less than 2 years.

The writer was assisted in this unusual work, bristling with new problems and difficulties, by O. H. Ammann, M. Am. Soc. C. E., as Principal Assistant Engineer in general charge; D. B. Steinman, M. Am. Soc. C. E., on computations of superstructure; and W. A. Cuenot in the drafting and checking of detail plans. R. T. Robinson, Assoc. M. Am. Soc. C. E., acted as Resident Engineer, and R. E. McGough as Chief Inspector of Steelwork at the shops.

The foundations and masonry were satisfactorily executed by the Dravo Contracting Company of Pittsburgh. The steel superstructure was fabricated and erected by the McClintic-Marshall Company of Pittsburgh, under its Chief Engineer, the late Paul L. Wolfel, M. Am. Soc. C. E. The erection was in charge of E. A. Gibbs, Assoc. M. Am. Soc. C. E., General Manager for McClintic-Marshall Company, with Mr. Clyde B. Pyle as his Field Engineer.

The writer desires here especially to acknowledge the conscientious and painstaking labor of his assistants and the helpful experience of the contractors, who combined in the successful completion of the work.

DISCUSSION

C. A. P. Turner,* M. Am. Soc. C. E. (by letter).—This paper on the Sciotoville Bridge, coming as it does from an engineer who has earned his place in the front rank of the Profession by the design and execution of one of the finest examples of long-span bridge construction—the Hell Gate Arch Bridge—will be received and read with unusual interest. The structure described is pleasing in appearance, satisfactory from the standpoint of rigidity and safety, and was erected in a creditable manner without mishap.

The span is somewhat in excess of those common in simple bridge truss design; but the fact that it is a double-track structure, and is wider and of correspondingly greater weight than a single-track railway bridge or the ordinary highway bridge, should extend the economic span length for the simple truss up to 850 ft. at least.

As an advocate of the continuous bridge, Mr. Lindenthal does not limit its application to long spans in place of the cantilever, but contends that it is economical for spans customarily regarded as solely within the province of the simple truss span. In assuming this position, he has raised fundamental questions regarding the underlying considerations affecting the economy of truss design.

The disadvantages of the continuous structure have been reasonably and fairly treated in the paper. They should not prevent the adoption of the type, providing economy results. However, if the economy claimed by the advocates of the continuous truss is unsubstantiated by the application of mechanical laws governing economic proportions and form, then the field of the type is limited to long spans which, designed with economic depth for vertical loads as simple spans, would be top-heavy under the overturning moment of wind pressure.

In the now classic controversy between Mansfield Merriman, M. Am. Soc. C. E., and Charles Bender, on the relative merits of continuous *versus* simple span bridge structures, Merriman as a strong point predicated the economy of the continuous truss as against the simple truss on the known economy of continuous beams of uniform section. Bender claimed that he could design a series of simple trusses, which would meet the requirements of a given specification with less metal than his opponent could design a continuous bridge covering several spans.

It would seem that the analogy between the economical relations of the continuous beam to the simply supported beam is inapplicable to the relations of the simple *versus* the continuous truss frame, because the web is of constant section in the beam, whereas in the truss it is proportioned to meet the requirements only of the variation of shear along the length of the span.

Again, although the claimed ability of Bender to design simple trusses of less weight than the continuous frame proposed by Merriman, might decide the question of ability of the contestants, it would not necessarily settle or explain the principle at issue.

* Cons. Engr., Minneapolis, Minn.

As in the case with the beam, the truss frame is called on to support the moment tending to elongate the bottom chord and shorten the top chord horizontally, and the shear or cross-breaking vertical force. The total weight of the frame is the sum of the weights of the members designed to support these different forces added to the weight of the floor system and lateral bracing.

Simple Bridges vs. Continuous Bridges—Parallel Chords.—With parallel chords, the cross-section and the weight of the chord decrease as the depth of the frame increases, and, conversely, the weight of the web increases as the depth increases. Therefore, in such a frame, the economic weight approaches a minimum when the weight of the web equals the weight of the chords, and this relation holds true whether the frame is continuous or simple.

Economic Depth Greater for Simple Span.—Continuity renders the live load shears resisted by the webbing somewhat greater than those in the simple span. Therefore, the web of the continuous span would be heavier than that for the simple span, but the maximum moments are smaller for the continuous span. If it is a through span with inclined end posts, the chord length is shorter in the simple span than in the continuous span and if, for this reason, the chords were assumed to be equal in weight with trusses of the same depth, then, because of the difference in the webbing, the economic depth of the simple span would of necessity be greater. Parallel chord bridges, however, are not economical for even moderately long spans and, for these trusses of variable depth, there is a more radical difference in the relative economic depth than for the case of parallel chord bridges of the two respective types.

Economic Depth—Non-Parallel Chords.—Simple Spans.—The reverse law of summation of shears and moments is most favorable to the simple span. Thus, shear increases from mid-span to the support, and applied moment increases from the support to mid-span. Accordingly, the simple span truss may be made deep at the center and of small depth at the end. With the greater depth at the center where the shear is the least, the web members, although long, are light. With decreased depth toward the support, the heavy web members are short, so that by this arrangement the weight of the web becomes greatly reduced over the case of parallel chords. Moreover, because of the inclination of the chord, the inclined chord functions in a dual capacity, resisting both shear and moment, still further reducing the weight of the web which, with the parallel chords, was one-half the total truss weight.

Contrast this reverse law of summation, so helpful to the economy of the simple span, with the conditions which are unfavorable in the continuous span. The moment is greatest numerically over the support where the shear is greatest and to obtain depth to resist this moment, heavy web members become long in place of the correspondingly short members of the simple span resisting the same shear. The economic depth for the simple span thus becomes from 20 to 30% greater than for the continuous truss for more moderate spans with reduction, in view of overturning moment of wind for the longer spans.

As, with economic height, the web of the simple span would reduce to about one-third the weight of the continuous type, and the chord lengths would be less, the conclusion appears inevitable that instead of the economy

claimed for the continuous frame, it is generally lacking in economy to the extent of 30 to 35%, as compared with the simple truss frame for moderate spans. The question of the economic frame or truss is only a part of the problem of economic bridge design. The lowest total cost is the object that the progressive engineer strives to attain, but rarely achieves. As he approaches it from a consideration of the minimum truss weight, other factors appear, which are antagonistic to the triangulation he has fixed on, for truss economy. It may be said that although the economic theory of design may appear to be very simple in theoretical works on bridges, in practice, it is otherwise.

The truss triangulation which gives apparently the least weight for the truss, commonly gives difficulty in obtaining desirable panel lengths for the floor system; the stringers are either too long or too short; there is always something that does not fit, and the resulting solution is a series of compromises to approximate the desired end. Again, in the longer span bridge, if the depth is economic for vertical loading, it will be ill proportioned to resist the overturning moment of severe storms, unless the floor is wide and the work heavy.

Viewing the Sciotoville Bridge as a continuous truss span, the writer believes Mr. Lindenthal has approached closely to the proper economic depth. His view that the steel weight is 20% less than that of a properly designed simple span is mistaken, and the writer would substantiate his view by the submission of Fig. 25, which shows the proportions of the Sciotoville Bridge, together with a suggested form of simple truss span which, approximate computation indicates, would be lighter than the continuous design by the margin noted. The moment over the support and the moment at the center of the continuous and simple span design would be equal. As the simple span is about 10% deeper, the maximum cross-section of the chord will be nine-tenths as great in the simple span. As the simple span design is such that advantage is taken of the reverse summation of shear and moment to an extent impossible in the continuous span and, because of the greater inclination of the chord members in the latter, the web proper would be about one-third of the weight of the web of the continuous span and, again, the total weight of the chords would be less, because of the shorter length, and it would appear that, as far as the truss frame is concerned, there should be a difference of at least 35% in favor of the simple span as against the continuous frame.

Consider the floor system: As the panel length is shorter, the stringer moments will be approximately six-tenths as great for the simple span, and the dead load moment would be one-half as great. The floor-beams would be increased in number, but reduced in cross-section. The number of main joints in the truss would be few, with longer members to handle, thereby reducing details. Wide variation in the make-up of the top chord and end post of the continuous type would be avoided, and simplification of details would result from greater uniformity of sections. The breaking truss to stiffen the floor-beams laterally would be eliminated automatically and the lateral system reduced in weight by connecting the laterals at the center of alternate floor-beams. Lateral stiffness would be secured by full triangulated

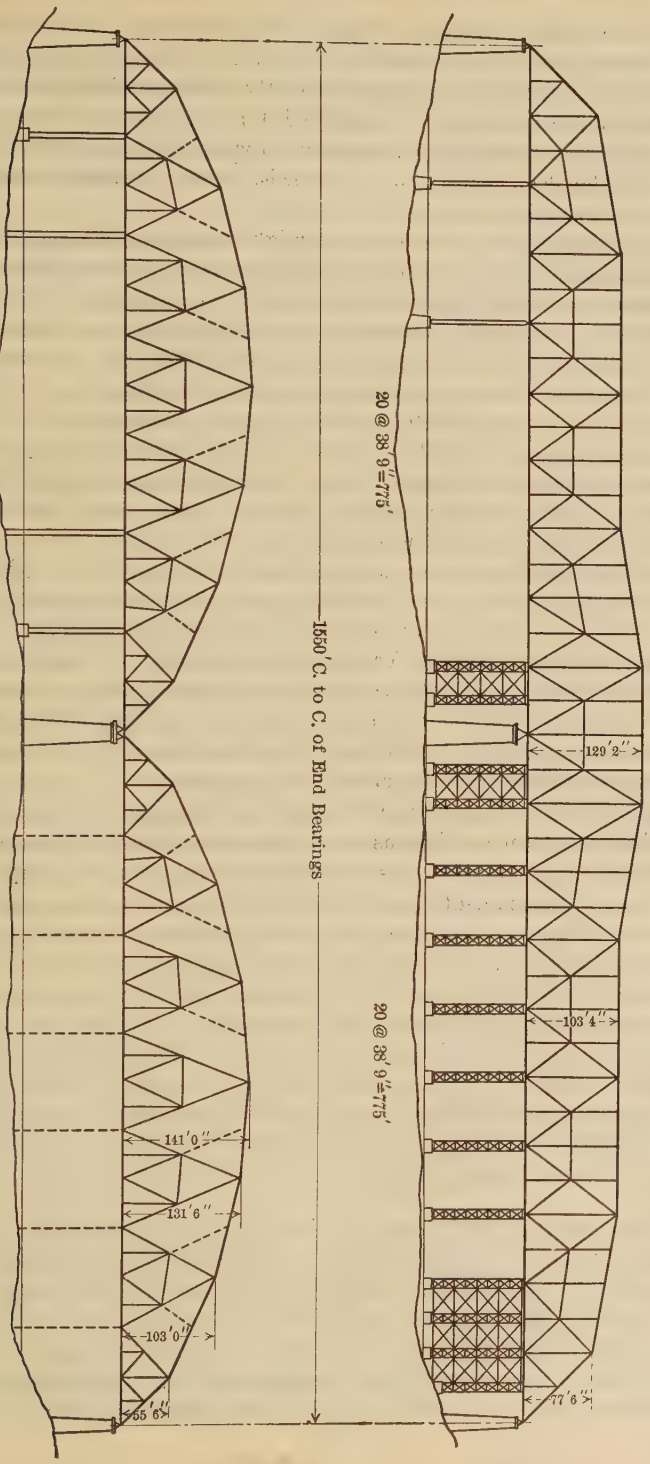

28 @ 27′ 7½″=773′ 6″
20 @ 38′ 9″=775′
1550′ C. to C. of End Bearings
20 @ 38′ 9″=775′
28 @ 27′ 7½″=773′ 6″
129′ 2″
103′ 4″
77′ 6″
141′ 0″
131′ 6″
103′ 0″
55′ 6″

Fig. 25.

bracing on a smaller number of planes than the portal bracing used, with a corresponding reduction in weight. A batter of the trusses of 5 ft. from the vertical in the total height would add to the lateral rigidity and permit some reduction of section otherwise required for top lateral and chord bracing. The number of members which perform no work as truss members, other than supporting the dead weight of the chords, would be greatly reduced in the suggested simple span.

The simple truss design suggested might come somewhat outside the pale of precedent which Mr. Lindenthal commends in the work of the early builder, but, because it is practical, it should not be considered a criticism of the serviceable and excellent structure described in this interesting paper. Approach to the goal of ultimate economy, for which all engineers are working and may never expect to reach, must create precedent.

Erection.—In order not to exceed the cost of erection of the continuous bridge, the method of erection of the simple span would likewise need to follow somewhat unprecedented lines. The fact that the river bottom is of sound rock would make the problem much simpler than if dealing with the Missouri River where there is from 50 to 60 ft. of shifting silt. Leaving a wide channel under one span, as was done in the erection of the Sciotoville Bridge, would necessitate the cantilever erection of three main panels of one span, which would not appear to present any especial difficulty.

Although opinion may differ on the relative cost of erection of the two types, of the quantity of steel required for the same specified working stresses, any difference of opinion is readily solved by working out the designs and calculating the weights, if the approximate solution by the application of mechanical principles is considered open to question.

Many American engineers have made as conscientious an effort as Mr. Lindenthal to design continuous truss bridges economically, impelled thereto by the mistaken assumption of some proportionate economy comparable to the divergence of the moment areas of the two respective types, only to realize, more or less, the heavy handicap imposed by the reverse law of summation of shear and moment favorable to the simple span and unfavorable to the continuous truss. In the comparison of the cantilever and continuous type, the moment areas approach identity for comparable depth and section over and between supports, and, therefore, it would appear that the discredit Mr. Lindenthal would cast on the cantilever truss should be focused rather on a degenerate section typified by all web and no flange, toward which the make-up of the compression members in many designs has carelessly approached and, in the unfortunate instance of the cantilever, the ultimate danger line had been passed.

In the foregoing discussion, the consideration of the web as that part of the truss between the external chords is a departure from the comparison of the equality of the web and chords in the parallel truss bridge. It would seem, therefore, that the inclined chord or end post might be apportioned as web or chord as the sine and cosine, respectively, of its angle of inclination. On this basis, a fair mathematical apportionment might be realized, an equalization from which reasonably satisfactory conclusions could be drawn as to

the economic depth in the types under consideration. Because there is very little inclination in the chords of the continuous bridge, the apportionment suggested immediately indicates greater economy, as the hypothenuse of the triangle is less than the length of the other two sides, the sine and cosine of the angle of inclination, another viewpoint is had, from which comparisons and conclusions may logically be drawn.

The theory of economic truss proportions is too complicated to be embodied in simple mathematical form. The suggestion of Mr. Bender, that the best method of comparison of any two designs of similar types, without going into details too completely, was by multiplying the length of the members by the maximum stress thereon, and the sum total of the products should be a minimum for the economic frame, appears in the light of a good approximation, providing the uniformity of sections and such lack of wide variation as occur in the change of sign and intensity of the chord stress presented by the continuous truss, in comparison with the constant kind of stress and relative uniformity presented by the arched chord simple span. The wide variation in the stress of the chords of the Sciotoville Bridge undoubtedly accounts for the uneconomical thickness of the webs of the sections, their lack of breadth in proportion to the height of the truss, and that apparently unavoidable compromise in the make-up from which the designer would be free in treating the simple truss span.

If engineers are to consider the Sciotoville example as a riveted truss of unprecedented dimensions, they should not forget the old Forth Bridge with clear spans of 1 700 ft., and its riveted construction throughout. Riveted construction in simple span design has the inherent advantage that rolled shapes which make up the members can be secured in very long lengths and so incorporated in the structure, reducing the number of joints necessitated by pin-connected chords, because of the facilities for fabrication and the necessity of making the joints at panel points for the proper connection of the laterals, which is immaterial when one is dealing with riveted sections.

The suggestion of a slight inclination of the truss might be objected to on the ground of difficult lateral connections, but this objection is more imaginary than real, as was demonstrated in the St. Croix River Arch.* Like Mr. Lindenthal, the writer appreciates the added advantage from the standpoint of stiffness which some indeterminate structures present, but this may be secured with the indeterminateness of the frame largely reduced for temperature and settlement, as was done in the structure noted. The adoption of the riveted sections throughout in the simple span permits partial erection as a cantilever, adapting the design to conditions under which the pin-connected frame is at a disadvantage.

It is generally assumed that the continuous bridge has some inherent advantages from the standpoint of stiffness, as compared with the simple span. If, however, the economic forms, and maximum deflections are compared, this will not be found to be true, as maximum deflection with the continuous bridge occurs when some of the spans are loaded and others un-

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1.

loaded, and with its smaller depth at mid-span, with partial restraint only at the support, and with its smaller chord section, the maximum deflection will be found to be greater than with the economically designed simple span. The difference, although in favor of the simple span, is insufficient to offset material economy did that, in fact, pertain to the continuous bridge.

Diverse Laws of Economic Chord Inclination in Simple and Continuous Spans.—As the apportionment of the efficiency of inclination of the inclined chord in the simple span is as the cosine of its angle of inclination in resisting horizontal deformation of moment, and the sine of its angle of inclination in resisting the vertical deformation of shear, its efficiency in resisting shear and moment is as the secant of its angle of inclination is to unity, compared with horizontal chords on a unit weight basis.

Because of the reverse sign of the bending moment in the cantilever parts of the continuous span, there is a reversal in this law of economic efficiency. Take, for example, the member of U_{18} to U_{14}, in the author's continuous bridge design. For negative moment, its efficiency in resisting horizontal moment deformation is as the cosine of its angle of inclination. Its efficiency in resisting vertical shear is likewise as the sine of its inclination, but it carries that shear vertically a longer distance upward from the point of reaction, L_{20}, thereby increasing the material necessary to support it, instead of decreasing the material necessary to support the shear, as is the case with the inclined chord where the moment is positive. Accordingly, the cross-section of U_{18}–U_{14} presents a total efficiency in resisting combined shear and moment of an amount equal to the cosine of the angle of inclination instead of the secant of the angle of inclination, as is the case with the inclined chord of the arched simple truss span.

It is true that this inclination reduces the section required in the member, U_{18}–U_{16}, so that the economies of inclination are the algebraic sum of the plus and minus savings instead of the consecutive positive summation, as with inclined chords in the simple truss span.

The Latitude of Compromise of the Divergent Economic Requirements of Floor and Truss Panels Unfavorable to the Continuous Span.—In the foregoing discussion, it has been noted that truss triangulation frequently conflicts with economic floor paneling. Thus, comparing from the standpoint of the simple span suggested, the bending moment of the four lines of stringers is 70% greater in Mr. Lindenthal's continuous truss than in the simple truss diagram suggested by the writer. The dead load bending moment is about 100% greater; whereas, conversely, beam shear and bending moment of all the beams is approximately 10% greater for the simple span than for the continuous span partly offset by the greater dimensions laterally of the chords which would naturally be used. Accordingly, the range for compromise adjustment is the difference between one clear dimension for the simple span against two shorter dimensions, that is, the cantilever arm and the suspended span in the continuous truss of two spans, or three, if the span is an intermediate one, with an economic location for the point of inflection inharmonious with favorable panels for either cantilever, suspended span, or floor.

This is a practical disadvantage of no small importance, which should not be overlooked in a theoretical survey of the two types of construction. General experience with continuous draw-spans gives the practical engineer a better basis for a comparison of the stiffness of two-span continuous structures with single span structures of the dimension of the length of the arm, than the relations of bending and moment forces presented in works on bridge design, and from this experience conclusions regarding stiffness are diametrically opposed to those customarily drawn from incomplete theory of resistance based on moment and neglecting shear.

Among engineers, experienced in the art of bridge construction, few are egotistical enough to assume ability to design the most economical frame possible to devise, yet it would seem that a thorough discussion of theoretical principles should lead to less divergence of opinion. With theoretical treatises on continuous bridge construction estimating from moment distribution an economy of 25 to 40% in the material required for the continuous truss bridge, and practical experience and conscientious design apparently indicating a lack of economy of 35% for a two-span continuous structure to 25% for a four-span continuous structure, from consideration of essential provision for combined shear and moment and the relations indicated by the constant of the moment magnitude equations, as well as numerical values of the three-moment equations, there appears indeed ample room for harmonization of book theory and practical experience as it is viewed by those who take the negative side of the economic question and ample room for explanation on the part of those who assume the affirmative side. Undoubtedly, the author's closing discussion in the affirmative will be interesting and instructive.

In closing this discussion, the writer thanks Mr. Lindenthal for the presentation of a most interesting and valuable paper, which skillfully treats of many practical questions and discloses meritorious details in advance of current practice. Preference for the continuous bridge of moderate span differs from the majority opinion of American bridge engineers, because of lack of demonstrated economy on a scientific mathematical or design basis.

T. Kennard Thomson,* M. Am. Soc. C. E.—Reference has been made to the danger of placing falsework in the Ohio River, which reference is fully appreciated by the speaker, as he was Engineer of Bridges for the Ohio River Division of the Norfolk and Western Railroad when, in 1890-91, the Kenova Bridge was built a few miles above the site of the Sciotoville Bridge. At Kenova, the records showed a low-water depth of 6 ft. and a high-water depth of 106 ft.

The author has called attention to the desirability of rock foundations for continuous girders. Any departure from this fundamental should only be adopted after the most careful consideration. For example, a sufficient number of piles might be driven to carry safely several times the designed load; but unexpected contingencies may divert a river in such a manner that these foundations would be seriously imperiled by undermining. Again, the teredo, which had never been known in the locality before, might obtain access to the piles.

* Cons. Engr., New York City.

Some years ago, it would have been considered safe to cut piles off 30 or 40 ft. under water, as the teredo was only supposed to find access to the piles between high and low water. About twelve years ago, however, the speaker was retained to report on a highway bridge at Fall River, Mass., where one end of a pier had dropped 2 ft. in one night. A pile head was cut off and brought up with live teredo and also limnoria, both actively boring.

The piles had been cut off 30 or 35 ft. below the surface of the water and 4 ft. of timber grillage, carrying masonry pier, had been sunk on top of these piles. The bridge had been completed just two years prior to the collapse of the pier. As the piles were cut off at such great depth, and as there was a sewer discharging within 150 ft. of the pier, it would have been considered safe against the ravages of the teredo.

The speaker, however, has never been able to satisfy himself as to whether the teredo and the limnoria obtained access to the piles after they were driven, or while they were floating on the surface of the water before being driven. In any event, great care should be exercised against allowing piles to float in teredo-infested water before driving.

As the Fall River Bridge did not have continuous trusses, no serious damage was done. A coffer-dam was built around the pier and concrete was forced under the grillage around the old pile-caps and brought up outside the pier, above the timber grillage line, to form a new support for the bridge and to prevent any further damage by the teredo. The steelwork was then jacked back to its original position on a new bridge seat.

Charles Evan Fowler,* M. Am. Soc. C. E. (by letter).—The present era of bridge building may well be termed the long-span bridge era, as it is now possible to span wide rivers, estuaries, or canyons more scientifically and economically than at any past, or, perhaps, any relatively near future, period. The Sciotoville Bridge is a striking example as to what may be accomplished by the use of continuous bridges. It is the longest of that type ever constructed and now gives to America the proud distinction of having the longest spans for every type of bridge construction, namely, the Sciotoville Continuous Bridge, the Hell Gate Arch, the Quebec Cantilever, the Williamsburgh Suspension, the Metropolis Simple Truss Span, and the Willamette River Draw-Bridge.

The piers of the Sciotoville Bridge are illustrative as to what should be done in the reinforcing of monolithic concrete piers, not only for the prevention of cracks, but as providing a strong protective shell for resisting the impact from ice, logs, scows, or collisions from steamboats. For this purpose, the writer's practice has been to use wire cloth, placed 3 or 4 in. inside the surface of the pier, and it is economical and effective. The care exercised in obtaining a proper foundation for these piers is to be commended, as it is a fundamental and vital matter in the building of long-span bridges.

The appearance of the bridge is peculiarly pleasing when one considers that it is a two-span structure, with outlines formed by straight lines. This satisfying feature is due to proper depths of trusses and a strictly symmetrical struc-

* Cons. Engr., New York City.

ture. The system of trussing adopted is, also, without question the best that could have been selected and one well adapted to meet the problems that arose in the method of erection. These results were evidently achieved as a result of a trained judgment and careful investigation. It is no surprise, therefore, to know the final outcome as to the economy and efficiency of the structure.

The unit stresses used in making the design should receive careful study by the Special Committee on Specifications for Bridge Design and Construction, and all engineers having to do with the design of steel bridges of any magnitude, as engineers can no longer be so wasteful of the resources of Nature as they have been in the past. The loading adopted undoubtedly will prove heavy enough for the future, as the uniform train load was the governing one for the main truss members, and with the margin allowed inside the elastic limit of the material, the floor and primary truss members will probably never be wanting in strength. The use of riveted construction throughout is to be commended, as the structure in consequence thereof is much stiffer and more lasting, thus nullifying the slight decrease in cost that the use of eye-bars would have effected.

The salient points of this structure, as well as of this type of bridge, have been so well covered in the paper that the writer will not comment on them, but engineers should study them carefully and endeavor to absorb those larger problems of design which have been solved so well.

J. E. Greiner,* M. Am. Soc. C. E. (by letter).—The author mentions those early engineers who had the "genius that originates as distinguished from routine which merely imitates." It may be said also that in every structure of importance designed by Mr. Lindenthal there is evidence of this genius which originates. Each of his structures is practically a new creation as compared with the routine and stereotyped bridges throughout the United States.

The Sciotoville Bridge is no exception. It is a daring and handsome structure, decidedly "Lindenthalic" in all its features, and designed to carry Cooper's E-60 loading. No one will question its strength or its adequacy for any load that will ever pass over it, but one may well wonder whether this bridge designed for E-60 loading can be rated as an E-60 structure. The customary rating of railroad bridges is based on the ratio of the working stress to the ultimate strength, which ratio according to the American Railway Engineering Association specifications is 16:60, or 0.267. The ratio used by Mr. Lindenthal is 20:66, or 0.303, which is 13.5 higher than the standard and places this structure in the E-53 and not in the E-60 class.

The author's compression as well as his impact formulas are quite at variance with general practice. He may be right, and all other engineers may be wrong. It is admitted that working stresses and column and impact formulas always have invited juggling, and probably always will be subjects for the engineer's dicta when he has such a prerogative. There are, of course, standards for these stresses and formulas, but no one would expect Mr. Lindenthal to give much consideration to such standards. He establishes his own standards.

* Cons. Engr., Baltimore, Md.

If he is correct in rating his Sciotoville Bridge as an E-60 structure, then all the railroad bridges built for such a loading in accordance with the American Railway Engineering Association specifications are E-68 bridges, and if the Association's E-60 bridges are really such, then the Sciotoville Bridge is only an E-53 bridge.

D. B. Steinman,* M. Am. Soc. C. E. (by letter).—The continuous truss is an excellent bridge type, offering decided advantages (under suitable conditions) over all other forms of construction. Its general adoption for fixed spans has long been retarded by prejudices based on erroneous notions; but the successful execution of several notable examples in the last few years (Sciotoville, 1917; Allegheny River, 1918; Nelson River, 1918; Cincinnati, 1922) has served to dispel these prejudices, and the continuous truss has become established as an important type in American bridge practice.

In comparison with simple spans, the continuous bridge offers the same advantages as the cantilever, namely:

1.—Economy of material.

2.—Suitability for erection of one or more spans without falsework.

In addition, the continuous bridge is superior to the cantilever in

3.—Rigidity under traffic.

4.—Less abrupt stress changes under traffic.

5.—Elimination of expensive and troublesome hinge details.

6.—Less extra material or hazard in erection.

7.—Safety of the completed structure.

The results of economic comparisons between continuous and simple spans will be materially affected by

1.—The length of spans.

2.—The system of web bracing used.

3.—The specification provision for reversal of stresses.

The relative economy of the continuous type increases with the length of span or, in general, with the ratio of dead load to live load. It is wrong to draw a general conclusion against the economy of continuous bridges based on a comparison for small spans (as Bender did in his book "Continuous Bridges" in 1876).

Furthermore, for a correct economic comparison, the most suitable system of web bracing should be assumed for each respective type; a Pratt or Petit system may be most economical for the simple spans, but a Warren system will generally yield the best results for the continuous truss.

Finally, the results of the economic comparison may be upset by the unscientific practice of imposing a stringent reversal clause; these reversal clauses as applied to the proportioning of main sections are relics of exploded fatigue theories, and should find no place in modern specifications for long span bridges.

A proper comparison with corresponding simple spans will generally show a substantial saving of material in favor of the continuous structure. It is

* Cons. Engr., New York City.

difficult or impossible to arrive at reliable conclusions in this question from abstract argument or theoretical considerations.

For purposes of comparison, the writer has made careful estimates of the weight of simple spans of 775 ft., based on detailed calculations of all stresses and sections. The live load was E-60 (on two tracks), and designs were made both for Lindenthal's specifications and for the American Railway Engineering Association specifications. The results were as follows:

Weight of two trusses, in pounds per linear foot:

1 550-ft. Sciotoville Bridge	12 880	= 100%
775-ft. Simple span (Lindenthal's specifications)	15 300	= 119%
775-ft. Simple span (A. R. E. A. specifications)	17 650	= 137%

Total steelwork, in pounds per linear foot:

1 550-ft. Sciotoville Bridge	17 100	= 100%
775-ft. Simple span (Lindenthal's specifications)	20 300	= 119%
775-ft. Simple span (A. R. E. A. specifications)	23 000	= 134%

The foregoing weights of simple spans do not include the extra metal which would have been required for the erection of the channel span by the cantilever method.

These figures bear out Mr. Lindenthal's claim of a saving of 20% in the weight of the steel by the adoption of the continuous type for the Sciotoville Bridge.

According to comparative studies made by Winkler, the saving for continuous bridges of two, three, and four spans is 16, 19, and 21%, respectively, when the span length is about 300 ft.; and 20, 24, and 28%, respectively, when the span length is about 500 ft.

Generally, however, the economy of material is a secondary consideration in the adoption of the continuous type, the deciding advantages being the convenience of cantilever erection and the increased stiffness of the structure.

The following conditions are particularly favorable to the economy and efficiency of the continuous bridge in comparison with other types:

1.—Long spans.
2.—High ratio of dead to live load.
3.—Good foundations.
4.—Piers of moderate height.
5.—Moderate truss depth.
6.—Spans approximately equal.
7.—Cantilever erection.

When the spans are long, the other requirements assume minor importance.

Both the economy and rigidity of the continuous type increase with the number of spans, but the gain beyond three or four spans is insignificant. Moreover, a larger number of spans would create difficulty in providing for expansion on account of the great length between expansion joints. Another objection is the greater number of supports at which jacking operations would be required during erection for the adjustment of the reactions. (In a two-span bridge, only one support out of three requires jacking adjustment; in a five-span bridge, four supports out of six would require jacking, and the opera-

tion would be more complicated.) For these considerations, the number of spans in a continuous group is generally limited to three or four.

In a two-span bridge, the requirements of economy as well as of appearance are best satisfied by making the two spans equal in length. In bridges of three or more spans, a symmetrical layout is also desirable for appearance and for shop economy. In a three-span bridge, the economic ratio of spans is approximately 7 : 8 : 7; but considerable variations from these proportions will not materially affect the economy. In a four-span bridge, the economic ratio of spans is approximately 3 : 4 : 4 : 3; but these proportions may also be varied considerably without materially affecting the economy.

In many cases, the span arrangement is determined by natural conditions of the crossing (or by the desire to utilize existing piers) rather than by economic or esthetic considerations.

The effect of possible pier settlement on the stresses in continuous bridges has been grossly over-estimated by former writers on the subject. In the case of the Sciotoville Bridge, according to the writer's computations, an excess settlement of 1 in. at the end support would produce a relief in the end reaction amounting to 15 000 lb. This is only 0.6% of the dead load reaction, or 0.3% of the total $(D + L + I)$ reaction. The simultaneous increase in the middle reaction would be 30 000 lb., which is only 0.3% of the dead load reaction, or 0.2% of the total $(D + L + I)$ reaction at the middle support. An excess settlement of 1 in. at the middle support would produce effects equal to double those just given, but opposite in sign. The middle reaction would be relieved 60 000 lb., or less than 0.4% of the total $(D + L + I)$ reaction at the middle support. It is evident from these figures that any ordinary settlement of the piers would affect the stresses in the structure to so small an extent as to be negligible.

A former objection to the continuous bridge was its static indeterminateness. With modern methods of design and construction, however, it is possible to know the exact stresses in a continuous structure for any given conditions; the uncertainties can be made as small as in simple spans and the extra labor of the computations is trifling in itself, as well as in comparison with the advantages to be derived.

The method of calculation developed for the Sciotoville Bridge was described by the writer in an article entitled "The Elastic Curve Applied to the Design of the Sciotoville Bridge".* The entire work of calculating the elastic curve by this method requires only 2 or 3 hours at the most. After the elastic curve is determined, the remainder of the design is essentially the same as for a simple structure.

In the case of the Sciotoville Bridge, three successive designs were made:

1.—Preliminary approximate design, treating the truss as a beam with constant moment of inertia.

2.—More exact design, allowing for the variation in moment of inertia but with the web members neglected.

3.—Final exact design, allowing for the variation in moment of inertia with the contributions of all the members included.

* *Engineering Record*, August 28th, 1915.

The sections obtained in the first approximation were used as a basis for the succeeding designs. The elastic ordinates for the three assumptions, also for the assumption of triangular variation of I, are compared in Table 2.

TABLE 2.—Comparison of Elastic Curves, Sciotoville Bridge.

Panel point (1)		$\frac{x}{l}$ (2)	Assumption I = constant (3)	Web members neglected (4)	All members included (5)	Assumption, triangular variation of I (6)
A	0	0	1.000	1.000	1.000	1.000
	2	0.1	0.875	0.871	0.876	0.855
	4	0.2	0.752	0.744	0.754	0.720
	6	0.3	0.632	0.623	0.632	0.595
	8	0.4	0.516	0.505	0.509	0.480
	10	0.5	0.406	0.393	0.392	0.375
	12	0.6	0.304	0.287	0.287	0.280
	14	0.7	0.211	0.193	0.192	0.195
	16	0.8	0.128	0.114	0.115	0.120
	18	0.9	0.057	0.053	0.055	0.055
B	20	1.0	0	0	0	0
Area *AB*			4.381	4.283	4.312	4.175
Area *BC*			0.619	0.717	0.688	0.825
Sum of areas			5.000	5.000	5.000	5.000

Table 2 is useful inasmuch as its values can be adopted for the preliminary designs of other structures, thereby saving time and labor. For a structure similar in general outline to the Sciotoville Bridge, the values in Column 5 should be used. For girders and trusses with parallel chords, the values in Column 3 would probably be a closer approximation.

The assumption of triangular variation of moment of inertia (assuming I to vary as the ordinates of a triangle from zero at the ends to a maximum at the intermediate support) may also be recommended as a basis for preliminary or approximate design. This assumption will generally represent the actual variation of I in a two-span continuous truss about as well as the assumption of constant, I; moreover, it yields results of striking simplicity. The elastic curves are parabolas, and other convenient relations obtain. Instead of the more laborious "Theorem of Three Moments", one is able to use the following simple formulas for the moment over the middle support. For concentrated loads:

$$M_2 = -\frac{1}{2} \Sigma M$$

in which M is the simple-beam bending moment produced by each concentrated load at its own point of application. For continuous or concentrated loads:

$$M_2 = -\frac{1}{l} (A_1 + A_2)$$

in which A_1 is the area of the simple-beam moment diagram in one span, and A_2 is the area in the other span.

In making preliminary assumptions as to distribution of dead load, variations of sections of truss members, and variations of moment of inertia, the corresponding distribution graphs for the Sciotoville Bridge may be of assistance. With this object in view, the writer has prepared Fig. 26 which shows, by graphs, the distribution of material in the Sciotoville spans.

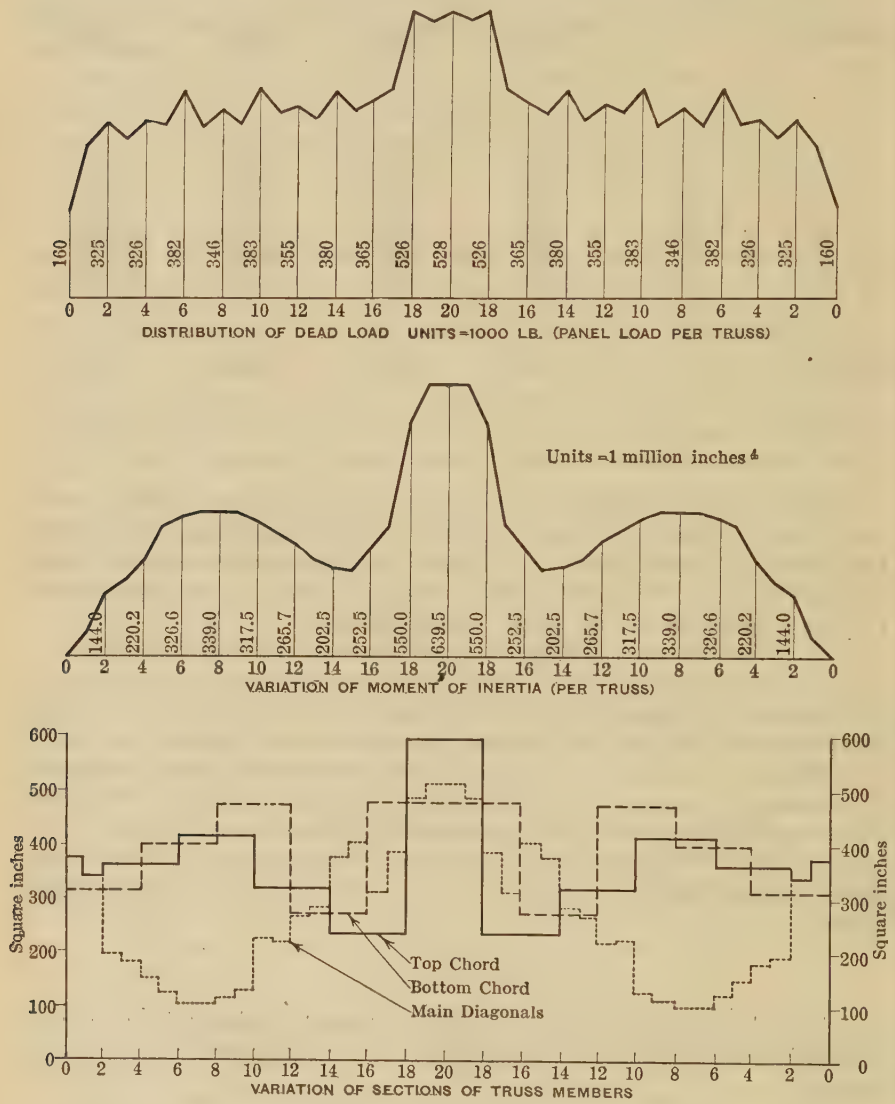


FIG. 26.

Certain improvements in distribution of material, with possible saving in total weight, may be secured by intentionally building the middle support lower than the end supports by a predetermined amount. The excess of middle

reaction over end reactions can thus be somewhat relieved, and the greatest negative and positive moments can be equalized. In a two-span girder of equal spans and constant, I, it will be advantageous to lower the middle support an amount defined by:

$$D = (1.9\,w + p)\,\frac{l^4}{145\,E\,I}$$

in which w = the dead load, and p = the live load per linear foot. This will reduce the negative moment over the middle support from about 16 to 31% (depending on the ratio of $w : p$) and will increase the maximum positive moment within the span to an equal value. For the Sciotoville Bridge, the necessary camber or lowering corresponding to this formula would be about 14 in. This would have reduced the maximum required section (over the middle support) from 596 to 520 sq. in., a reduction of 76 sq. in.; and it would have increased the mid-span section from 474 to 520 sq. in., an increase of 46 sq. in. Such equalization of maximum sections, with a reduction of 13% in the extreme heaviest sections, is an advantage worth considering in future designs.

In common with other indeterminate structures, the continuous bridge offers the possibility of varying the distribution of stress by adjustment. In the case of the Niagara Railway Arch Bridge,* it will be recalled how Charles Evan Fowler, M. Am. Soc. C. E., secured a favorable readjustment of dead load stresses by changing the thickness of the shims at mid-span. Similarly, in a continuous bridge, the dead load stresses may be redistributed by the predetermined raising or lowering of one or more supports. This may be done during erection, in order to reduce the maximum required sections, or in order to secure a more favorable distribution of material to resist erection stresses; or the jacking operation may be resorted to after the bridge has been standing for some time in order to relieve members which, for any reason, may be found to be overstressed.

Three notable innovations in the design of the Sciotoville Bridge were as follows:

1.—The adoption of U-shaped floor-beams, designed as inverted arches.

2.—The adoption of lattice sway-frames, instead of the usual rigid cross-frames.

3.—The initial bending of the members to neutralize the secondary stresses.

The adoption of the U-type of floor-beam is probably without precedent in practical bridge design, although the combination of conditions warranting its use may easily occur in any long span structure. The application of the U-shaped floor-beam is indicated in any structure in which there obtains limited floor depth with excessive width between trusses. This combination of conditions renders the straight type of floor-beam uneconomical; and the second condition, from considerations of clearance, renders the U-type practicable. Both these conditions obtained in the Sciotoville Bridge. In this structure, in order to secure adequate transverse stiffness against wind and lateral forces, the length of floor-beam, or distance, center to center of trusses, was made 38 ft. 9 in., or one-twentieth of the span. This is nearly 10 ft. in excess of the width required for train clearance, this excess of clearance being a

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1919.

consequence of the unusual length of span for a bridge of only two tracks. The excessive distance between the truss and stringers results in high bending moments, so that a floor-beam depth of 8.5 or 9.0 ft. would be desirable for economy. Actually, a depth of only 7.5 ft. was available. Although an additional 12 or 18 in. of height would have given lighter floor-beams and stringers, it could not be secured except at the expense of raising and lengthening a 40-ft. fill nearly 2 miles in length at the Kentucky approach. To overcome the disadvantage caused by this limitation of depth, the U-shaped floor-beam was devised. This design yields the following advantages:

1.—A material saving in the total weight of floor-beam.

2.—A considerable reduction in the bending moments, thereby avoiding some of the difficulties of providing extra heavy flange sections.

3.—Increased transverse stiffness of the bridge, the U-frame becoming an addition to the sway-bracing.

4.—Elimination of the secondary stresses in posts or hangers caused by the deflection of straight floor-beams.

5.—Elimination of the secondary stresses in straight floor-beams due to their partial constraint at the ends.

6.—Increased resistance of the structure to vibration.

In the Sciotoville Bridge, the adoption of the U-shaped design, according to the original comparative estimate, reduced the maximum live load moments by 39% and effected a saving of 9 000 lb. of steel in each of the thirty-nine intermediate floor-beams. This saving in weight may be offset somewhat by increased unit cost of the shopwork, but the other advantages alone would justify the adoption of the U-shaped floor-beam wherever adequate lateral clearance is available.

In analyzing the stresses in the U-frame, it is important to make proper allowance for the shortening of the bracing strut which takes up the horizontal thrust of the inverted arch.

The use of ordinary sway-bracing of the rigid type was avoided, in the design of the Sciotoville Bridge, on account of the high stresses which such bracing would suffer under one-sided loading of the bridge. Under the extreme case of one track of one span and the opposite track of the other span being fully loaded, there will be a difference of 2 in. in the mid-span deflections of the two trusses. To permit this unequal deflection without rupture, the usual sway-frames of rigid type were replaced by lattice-frames combining strength with elastic stiffness.

The scheme for neutralizing the high secondary stresses in the structure is another feature adding distinction to the Sciotoville Bridge. The necessary erection operations were calculated with unprecedented care and thoroughness, and the measurements of jacking forces and deflections in the various erection stages afforded an excellent check on the calculations.

The Sciotoville Bridge is a striking example of scientific design. It represents an unusually intensive application of engineering theory and resourcefulness to the determination of the most efficient disposition of the material of a structure, and to the rigorous advance planning of every step in the operations of fabrication and erection.

HENRY H. QUIMBY,* M. AM. SOC. C. E.—The speaker recalls that forty years ago the computers of a certain bridge company made simple spans of the two plate-girder spans of a railroad bridge because of want of confidence in their figures and fear of some indefinite consequences if the girders were made continuous over three supports.

Continuity of structure is an economical and advantageous principle of design and, if intelligently used, is practical and safe. In bridges of several arched spans, it has been applied with satisfactory results—in one case of five spans, the steelwork was both the reinforcement of the concrete arch ribs and provision for the erection of both the steel and the concrete without falsework. In another bridge that had a large number of rod-reinforced concrete arches, reinforcing rods were embedded over the piers as the tension members to constitute the first voussoirs cantilevers for carrying the forms for successive voussoirs. The rods were left exposed for a few inches, and after erection was complete they were cut by sawing. The tension was so great that each rod snapped apart when partly cut. In later bridges, both with structural shapes and screw rods as tension members over the piers, the ties were permitted to remain, as being not only harmless, but possibly contributory to stiffness as well as to stability.

In the Sciotoville Bridge, the effect on the stresses in the members, which the paper states would be produced by deformations that would attend certain settlements of piers, is surprisingly small in view of the great height of the trusses. As so much settlement can hardly be imagined, the continuity of truss is certainly justified.

The method of erection and the means adopted to minimize secondary stresses are interesting. Both must have been expensive, and some information regarding the cost of the work would be instructive. The plan of erection was dictated in a measure by considerations of safety under the attendant conditions of the site and was justified by its success. The method adopted for reducing secondary stresses was bold and scientific, but must have been very costly, and it is understood to have involved a great deal of heavy drifting and reaming to get the holes to admit the rivets. Can the author give any facts regarding these points, and any comparison of the extra cost of the work as done with the cost of providing additional section at the critical points of members affected by secondary stresses?

GUSTAV LINDENTHAL,† M. AM. SOC. C. E. (by letter).—The contention made by Mr. Turner, that simple girders may be designed with no more or, perhaps, with less steel than continuous girders, is based on the same incomplete grounds often asserted before. Short panels to reduce floor weights, great height between chords to reduce chord sections, and curved chords to reduce web stresses, are all long known devices for attaining economy. They can be used in both the simple and the continuous types, and also in the cantilever type. The question of economy can hardly be settled by a comparison of strain lengths in the several types. The experience of bridge designers during the

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† Cons. Engr., New York City.

last 30 or 40 years has fairly well established the best proportions for each type.

The theory of strain lengths originated by Charles Bender (who was personally known to the writer 45 years ago), seemed at the time a plausible basis for estimating and comparing the economy of bridge and roof trusses, fortified as it was by an elegant deduction from Clapeyron's theorem of deflections. With simple elements of loads and strains, it can be useful; for the correct dimensioning of bridge members, however, a greater number and variety of stresses have now to be considered than were thought of in Bender's day. To the stresses from dead and live load must be added the various combinations with effects of impact, traction stresses, wind pressure, secondary stresses, etc. The theory then ceases to be useful, especially so in bridge frames having heavy members with reversible stresses (tension and compression), as in continuous and cantilever trusses.

No fact on the strength of steel is more firmly established from practical tests and after much theoretical travail than that steel and wrought iron will safely resist without deterioration a nearly infinite number of variations and reversions of stresses, provided the unit stress remains within, say, two-thirds of the elastic limit. To take advantage of this fact in economic bridge designing, the stresses must be accurately known. This is not always feasible with live load stresses, some of which are dynamic, but are, nevertheless, assumed to behave statically like stresses from dead or other quiescent loads. In some members, impact stresses may exceed the statical stresses from live load of the same or opposite sign. The theory of strain length under all these conditions becomes too uncertain for the *a priori* comparison of economy in designs. The only reliable comparison in any given case is on the basis of fully worked out stress sheets, complete erection schedules, and estimates of cost.

If two spans of simple girders with a height of $\frac{1}{5.5}$ of the span, as proposed by Mr. Turner, should show greater economy, which is doubtful, even with alloy steel, then there is no reason why greater height could not also be used for the same purpose in continuous girders; but would it be good designing? Engineers are expected to build good bridges economically. That does not mean freak bridges under the pretense of scientific economy which too often may be false economy.

In this double-track bridge, the great simple-girder height would be nearly four times the width between girders. A train passing over the bridge would deflect one girder more than the other by several inches. That would throw out the top of the girder nearly four times as many inches. It would be a very wabbly structure, possibly a so-called economic design, but inferior as far as behavior under use is concerned. The inclining of the trusses, as suggested by Mr. Turner, would not improve it.

In the continuous girders, the greatest height (also the most weight) is over the middle pier, where the girders cannot deflect horizontally or vertically. The wind bracing between the chords is also continuous. The four continuous girders form a long rigid box of great stability, open at the ends, exposed to the least amount of vertical and horizontal oscillation. In other

words, the continuous girders are in every way stiffer than the simple girders.

A floor with 28 panels per span, as proposed by Mr. Turner, in place of 20 panels, for the sake of saving an insignificant quantity of steel in the stringers, would require 40% more handling in the shops, with many thousand more field rivets, and 40% more field operations during the erection. The greater cost of field labor would cancel the inconsiderable saving in steel for stringers of shorter panels. In addition, the greater compactness of long panels is discarded. Owing to the lighter floor, the center of gravity of the single girder structure is higher and contributes another element to the wabbliness of the bridge. The top chord panels would be 50% longer than in the continuous girder and the cross-sections, therefore, 30% larger, thus effacing any economy from greater height of trusses.

The simple girders would require considerable extra material to be erected by the cantilever method over the large river opening needed during erection. It would also have required more time for shifting the falsework and another material yard on the Kentucky side, which was not available. Compact, short, well braced, compression members keep the chords rigidly in line, an advantage largely absent in the simple girder. The bridge is situated in a region subject to tornadoes which have wrecked one or more bridges over the Ohio River. The bracing of the continuous girders can be made better and stronger, and they are thus capable of offering greater resistance to wind pressure.

The gravamen of Mr. Greiner's discussion is that the capacity of the bridge is not really for an E-60 loading, but only for an E-53 or, say, 12% lighter loads, when based on the specification of the American Railway Engineering Association. The A. R. E. A. specifications are intended only for ordinary spans, not exceeding, say, 350 ft.; for longer spans they are wasteful in the trusses. This fact is confirmed by a comparison of a few sections as shown in Table 3, in which is given the sectional area of a number of typical members of the Sciotoville Bridge, as designed according to the writer's specifications in comparison with those required by the latest A. R. E. A. specifications and E-60 loading. Table 3 also gives a comparison of the strength of these members for the two specifications, based on the fact that the writer's specifications require steel at least 13% stronger than that called for by the A. R. E. A. specifications (minimum yield point, 35 000 against 30 000; ultimate strength, 62 000 against 55 000).

The statement by Mr. Greiner that the bridge has a live load rating of only E-53 is shown by Table 3 to be unjustified and is made without examination of the requirements of the writer's specifications other than the basic unit stress (20 000), which may mislead superficial readers.

In the floor system and floor hangers (as in short spans generally), the writer's specifications give sectional areas up to 25% in excess, and in strength of members up to 41% in excess of those required by the A. R. E. A. specifications. That makes the live load rating for the floor construction in this bridge really E-70 to E-84 on that specification. This is due principally to the greater impact allowance for short spans, resulting from the writer's formula for impact, which, as far as is known is the only impact formula

deduced from well-known facts on the durability of steel under impact conditions. It is simple of application and gives rational results for the shortest and the longest spans alike. This advantage adheres also to the writer's specifications, or "Rules of Design", used in the Hell Gate, Sciotoville, and other bridges. These specifications give well-balanced working stresses, insofar as uniform durability of structure is concerned, for the shortest and the longest spans, which is not the case with the A. R. E. A. specifications.

TABLE 3.

Member.	Existing area, in square inches, Lindenthal design.	Area, in square inches, required by A. R. E. A. specification.	Percentage of excess area, Lindenthal specification over A. R. E. A. specification.	Percentage of excess strength, Lindenthal specification over A. R. E. A. specification.
Stringer:				
Tension flange, net	26.66	24.0	+11	+25
Compression flange, gross	30.58	29.4	+ 4	+17
Floor-beam:				
Tension flange, net	42.63	34.1	+25	+41
" " , gross	46.32	41.8	+11	+25
Trusses:				
Top chord:				
U 6–*U*10, gross	435	518	—16	— 5
*U*10–*U*14, gross	317	374	—15	— 4
*U*14–*U*18, net	198	187	+ 5	+18
*U*18–*U*20, net	501	579	—13	0
Bottom chord:				
L 4–*L* 8, net	403	404.1	— 0	0
*L*12–*L*16, net	230.1	255.1	—10	+ 2
Diagonals:				
M 3–*L* 4, net	149.0	154.6	— 3.7	9
L 4–*M* 5, gross	150	152.6	— 1.6	11
U 6–*M* 7, gross	104.5	140	—34	—26
L 8–*M* 9, net	93	124.0	—33	—25
*U*10–*M*11, gross	224	258.0	—13	0
*U*18–*M*19, gross	486.7	655.0	—25	—15
Main hanger, net	90	85.3	+ 6	+19
Sub-hanger, net	49	44.2	+11	+25

The sectional areas of the bottom chord (loaded chord) of the Sciotoville Bridge are nearly the same, because in these members the impact allowance is more nearly the same, for both specifications, but the reduction in areas which might result from the higher unit stresses of the writer's specifications, is offset by the justified greater allowance for lateral, wind, and brake forces, which may co-exist in these chords with the dead load, live load, and impact stresses.

The A. R. E. A. specifications would require some sections of the top chord and main web members, to be 34% larger, and in strength to be 26% greater. This would be plainly a waste of steel due to the low unit stresses allowed by the A. R. E. A. specifications (maximum 16 000 lb. per sq. in. in tension and 12 500 lb. per sq. in. in compression), which low limits are not justified in a bridge of this size. The waste is also in part due to the absurd requirement of the A. R. E. A. specifications that members subject to reversal of stress shall be proportioned for the greater stress plus 50% of the smaller stress.

The A. R. E. A. specifications do not require sufficient metal for the floor structure and for short spans where it is necessary, because these structures are stressed to the violent maximum from nearly every train, and they require too much metal for the truss members of long spans, in which members, the maximum stresses are far from violent and rarely, if ever, occur.

The belief of Mr. Quimby that the method of reducing secondary stresses was costly and involved a great deal of heavy drifting and reaming in order to get the holes to admit rivets is not based on fact. There was no trouble or extra work on that account in the shop.

Although it is true that the bending of the members in the field, in order to make the rivet holes fit, required some special operations, the cost of such work was comparatively small and was fully justified by the advantage gained. In fact, the fabrication and erection cost of the bridge per pound of steel was remarkably low, and less than for pin-connected work, as proven by the bids received for both types of structures.

The discussion by Mr. Steinman is a welcome contribution on some details and features already mentioned, in the main, by the writer in his paper, omitting elaborations.

The conclusions by Professor Winkler, the German nestor of bridge mathematicians 45 years ago, mentioned by Mr. Steinman, on the economy of continuous girders, are fully confirmed in the Sciotoville Bridge. French and German engineers early showed much less hesitation in the use of that system than American engineers. Almost all those early European bridges are still in good condition, although railroad loads on them have increased, but not as much as in America.

Mr. Steinman's idea of reducing the height of trusses over the middle pier and increasing the height at mid-span, would not be, on the whole, advantageous or economical, as the questionable saving of metal in the chords would be outbalanced by increased sections and metal in the web members toward the ends of trusses. There would also be, in this case, the objection of greater erection stresses and too much deflection during erection, with greater secondary stresses in cantilevering.

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Paper No. 1497

WATER TRANSPORTATION*

A SYMPOSIUM

By Messrs. R. H. M. Robinson, Winthrop L. Marvin, Emory R. Johnson, and Samuel O. Dunn.

With Discussion by Messrs. Gardner S. Williams, J. E. Willoughby, John C. Trautwine, Jr., Augustus Smith, C. C. Vermeule, and B. F. Groat.

* Presented at the meeting of January 19th, 1922.

WATER TRANSPORTATION

By R. H. M. Robinson,* Esq.

There are about 60 000 000 gross tons of shipping in the world; 10 000 000 tons more than there were at the outbreak of the World War, and, at least, 20 000 000 tons, the speaker estimates, more than are needed to carry on the world's commerce. Directly and indirectly, actually or potentially, every ton of this 60 000 000 is a competitor of every other ton. That is a distinctive characteristic of the shipping industry—the great mobility of its productive factors. Ships are spoken of as productive factors, because from them emanate the services which are the merchandise of the ship owners.

A ship built for the New York-South American service does not compete directly with ships operating in the North Atlantic, but it may force another ship out of the South American service and into the North Atlantic. For this reason, it is wrong to say that foreign tonnage does not affect conditions in the United States intercoastal trade, because a ship built in Japan for trans-Pacific service may conceivably force an American ship from the trans-Pacific into the intercoastal trade.

This kind of world-wide competition is limited to a certain extent by the fact that special types of ships are much more useful in certain trades than in others; also, by the natural preference of operators to remain in trades where they are known and where they have built up useful connections. The fact remains, however, that there is a large amount of standard type tonnage on the seas, which is ready to do business wherever business may offer, and which, consequently, affects rates the world over.

To illustrate: The rates from the River Plate to the United Kingdom and the Continent, which were as low as 15/-, have showed a marked improvement and in December, 1921, reached a level of 32/6. The effect of these high rates was to draw tonnage that had been engaged in the less profitable Mediterranean trade, into the River Plate trade, and to induce British tonnage to load outward coal cargoes at low rates, in order to be in a position to pick up these attractive homeward freights. In this way, the improved cargo offerings at the River Plate were soon offset by increased available tonnage, and the rise in rates was checked. To cite another case: The once profitable trade between the Pacific Coast ports of the United States and Europe has been brought to a deplorable state by the rapid introduction of additional tonnage into the service.

Thus, the fact that there is an excess of tonnage in the world is significant to every ship owner. It means intense competition, scrambling for cargoes, rate-cutting, and unprofitable business, unless the situation is adjusted, either by reduction of tonnage or increase of trade. There has been wide discussion of the advisability of an international agreement for the scrapping of the present surplus of tonnage and the limitation of future building. Most ship owners, however, are holding on as long as they can in the

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hope that the adjustment will come through an increase of trade rather than a scrapping of tonnage.

The revival of trade will eventually come, but the speaker's views are that it will be gradual and that some years will pass before the serviceable shipping available will be given adequate employment. Meantime, the poorer ships and doubtless the weaker companies will drop out of the business, and those that stay in, will do so under conditions of keenest competition and with meager profits. Of course, freight services are referred to—passenger services require special consideration.

It is certainly not out of order to consider where American shipping stands with respect to this general situation and the long competitive struggle which is ahead of American shippers. Handicaps of a serious nature are suffered. American ships cost more to build, marine wages are higher, and American experience in the business is much more limited than is the case with their principal competitors. The speaker does not intend to go fully into this subject, but there are several points in connection with the last-mentioned handicap—the more limited experience of American shipping—which will be discussed briefly.

Long experience in a business means, among other things, the establishment of helpful connections and the creation of a certain good will in the markets served. In these traffic arrangements, the newer American shipping companies sadly lacked during their efforts to get started just after the armistice. They gradually are establishing these connections at home and abroad. Too much emphasis cannot be laid on the value of good agents abroad. It is a phase of the problem that is often under-estimated. American ships need good cargoes homeward as well as outward, if they are to yield profits to their owners.

Long experience has its benefits in operating efficiency, also. Efficiency and economy are problems in every industry, and nowhere more than in shipping. It must be evident that in a business which involves such great distances as in the case with a trans-oceanic shipping company, and in which unexpected conditions are always arising, inefficiency and waste are likely to play a prominent part. Although making no claims to exceptional talent in the eradication of these evils, the speaker's study of the problem has brought to his attention certain possibilities which may be worth mentioning.

Waste may occur with respect to materials and with respect to services. Dealing first with materials, one may look at the question of fuel, which constitutes a large item in the operating cost of a steamer. A boiler may be fed to a certain point with maximum results, and fuel that is used beyond that point is sheer waste. Generally, engine-room crews are alive to this fact, but often they carelessly or willfully disregard it.

Not infrequently it is found that ships are driven at an uneconomical speed, thus uselessly increasing the fuel consumption. It is a great temptation to a captain to push the ship to her speed limit in order to make port some hours, or a day, earlier. It takes the closest kind of checking and control by headquarters to eliminate this form of waste, that is, to determine what is the

economical speed of a given vessel for a given trade under given conditions, and to enforce the proper observance of that speed.

As regards food, it is recognized that men must be well fed on shipboard, but the wastage that is sometimes encountered is prodigious. Certain of the ships of the speaker's Company, which have always been run on what was considered an economical basis, are now costing the Company no more per man per day than they were before the war, although prices are about 50% higher. These results have been reached by giving the ships' stewards a maximum figure to which to work, and discharging him if he goes above that figure, without reasonable excuse. There have been relatively few discharges on this account. The men, moreover, are entirely satisfied with their food.

Just as a steward may waste food, so a deck or engine crew may waste paint, lubricants, and other supplies. It is a little more difficult to check up on this kind of inefficiency, but there is no excuse for allowing it to run wild. A common form of waste is that of letting machinery run down for lack of proper attention from day to day, so that, in the course of time, it breaks down entirely and requires extensive overhauling, and perhaps replacement of parts.

It is wasting services to carry a larger crew than is actually required. When business was good, the Company thought its crews were down to the practicable limit, but the pressure of hard times has brought about further reductions which, on some of the cargo vessels, have amounted to as many as six, seven, and eight men. This has been done, moreover, without handicapping or endangering the ship.

Services are being wasted when the crew is allowed shore leave to the extent of necessitating that shore labor be brought aboard to do their work. The American law requiring the payment of half the wages due, on demand of the seamen in port, induces the men to spend more time on shore than is good for themselves or the ship, and not infrequently leads to their being left behind, or to deliberate desertions.

The most extravagant form of waste is the wasting of the services of the ship. Several idle days at port, during which the vessel is earning nothing and is running up expenses at almost the same rate as if it was at sea, may easily destroy an otherwise fair margin of profit. Every hour counts when it comes to despatching a ship and it takes the concentrated, and one might almost say the consecrated, efforts of the operating department, crew, stevedores, and all others concerned, to eliminate losses from this cause.

The Operating Department of the speaker's Company has worked out a plan for consolidating at one port fuel delays which formerly had occurred at two ports. What the saving by this arrangement will amount to in the course of a year has not been calculated exactly, but it will doubtless run into many thousands of dollars.

It is self-evident that the elimination of waste in the operation of ships requires a high degree of co-operation as between management and men, and there are many plans for encouraging this. There is the idea of creating a competitive spirit among the men by comparing the records of different

voyages and of the various ships of a fleet. There is the idea of a bonus granted to officers and crew when a ship makes a particularly satisfactory voyage. Sick benefits and superannuation funds also work in this direction. In a large fleet, the promotion plan may be used effectively, that is, efficiency among the men may be encouraged by holding out to them the possibility of getting higher ratings with the Company as a reward for faithful service.

It is possible to go on almost indefinitely enumerating forms of wastage which should be closely guarded against. In a business as highly competitive as shipping, inefficiency and extravagant practices cannot be tolerated; and, as has been already stated, the necessity for economy is especially strong in the case of American ship owners. The speaker does not mean to say that a sort of super-efficiency will entirely offset the handicaps which American ship-owners suffer. He is convinced that a subsidy is necessary if American ship owners are to continue to operate under the American flag. Assuming, however, a subsidy sufficient to overcome the disadvantages growing out of peculiar American laws and conditions, ship owners still have a big problem before them in their effort to meet successfully the competition of foreign owners, because of the much longer experience of the latter in the business. The only possible way to meet this problem is to offset longer experience with more intensive study. Americans, in the engineering field particularly, have an aptitude for that sort of thing.

THE AMERICAN MERCHANT MARINE

By Winthrop L. Marvin,* Esq.

Few are the observers who would have ventured to predict in 1914 that one result of the great World War just opening would be to add 10 000 000 tons of ocean shipping to the merchant marine of the United States, thereby making this country a close rival, as far as total tonnage is concerned, of the United Kingdom which since 1860 had been the unchallenged "mistress of the seas". An expenditure of more than $3 500 000 000, the proceeds largely of several issues of Liberty bonds, has given the United States a merchant navy, measured by American methods, of 18 282 000 tons gross register in 1921, as compared with a total tonnage of Great Britain, measured by British methods, of 19 320 000 tons.

Of the new American merchant marine, however, this distinction must be made, that a great part of it is used in near-by home commerce, as, for example, the 2 839 000 tons of the great Northern Lakes and the mighty coastwise fleets of the Atlantic, the Pacific, and the Gulf of Mexico. However, American shipping registered for foreign commerce on June 30th, 1921, had attained a magnitude of 11 081 000 tons, an increase of 10 005 000 tons since June 30th, 1914, or a shipping twice as great as the entire great merchant fleet of the German Empire of the year when the Kaiser rashly challenged the world to arms. Of the actual sea-going vessels of the United States, in both foreign and coastwise commerce, 7 993 000 tons at the beginning of 1922 were owned by the United States Shipping Board, and 5 240 000 tons by private ship owners, firms, or corporations.

These are records of magnitude. The great merchant marine of our forefathers—the unmatched fleet of clipper ships and steam ocean liners—amounted to 5 353 000 tons in 1860. At last, after many years, American shipping has assumed "a place in the sun" comparable with other great National industries.

Of the merchant fleet of 1921, only 1 294 000 tons all told, out of a total of 18 282 000 tons, are sailing craft. The remainder, except for 1 180 000 tons of tow barges, is power driven. Allowing for the superior efficiency of the steam or gas-propelled ship—three times the efficiency of the sailing ship—the American merchant marine, measured by its capacity for transportation, has increased tenfold in sixty years. That is to say, the sea, lake, and river carriers of the United States have ten times the delivery capacity of the ships of 1860-61. This is a fact which justifies the merchant marine in holding its head reasonably high among the present-day great instrumentalities of transportation.

It speaks well for the essential vitality of the merchant shipping industry that on the coasts of the United States it has managed to increase its tonnage threefold and its actual capacity eightfold in six decades against the powerful competition of the most effective railroad systems in the world. For American coastwise ships and railroads are competitors in a considerable degree, although

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also co-operators with each other. An excellent example of the manner in which American railroads and American ships have been made to work together is the Southern Pacific Company, with its splendid fleet of 25 steamers, of a total tonnage of 129 938, plying between New York City and the Gulf of Mexico, as an essential link in one of the greatest of the transcontinental systems. It used to be an American policy, and a wise one, to yoke ships and railroad lines in a common service, and although this policy was largely forbidden by the Panama Canal Act of 1912, it is not easy to convince thoughtful men that the United States has been the gainer thereby.

The ships need the railroads and the railroads need the ships. If there cannot be common ownership, there must be at least close harmony and co-operation. It is a fact of much significance to the railroad systems of the United States that under the American flag there exists at present an immense fleet of ocean carriers, American-owned, and pledged to serve first American interests. With proper "team play" there is no reason why these American fleets may not be made dependable extensions of American railroads. Under a proper policy of National encouragement, and not discouragement, there is no reason why American shipping companies, with alert traffic organizations abroad, may not bring to American railroads a more efficient service than the foreign steamship companies have ever given, plus the vital factor of American good-will. If the plans of the present National Administration take root, no American railroad management will have a shadow of an excuse to link itself, by preferential traffic or terminal agreements, with a foreign merchant shipping, which is to say, the naval reserve, of any foreign Government.

Those interested in and of the American ocean-shipping industry bespeak from the railroads a closer understanding than has ever existed before. About one-third of the present ocean-borne tonnage is coastwise; about two-thirds normally and constantly must be working overseas. These coastwise carriers, having a natural advantage in not being compelled to maintain rights of way of stone, earth, and steel, have an important economic part to play in assembling cargoes at American ports for the overseas carriers, and in assembling and distributing goods which the railroad, in turn, must serve. One of the most powerful arguments for the strengthening particularly of the overseas merchant marine is that a more effective co-operation can be maintained by American railroads with such a merchant marine than would be possible with foreign corporations owned, controlled, and directed abroad, and maintained for the purpose of serving, primarily, alien competitive interests.

American shipping, overseas and coastwise, is suffering severely from the nightmare of depression that has seized the commerce and finance of the world. Of 1 300 steel steamers of the Shipping Board, fewer than 400 are now in operation. From one-fourth to one-third of the tonnage of private ownership is unemployed. This is not strange, with the exports of the United States shrunken to one-half the volume of the exports of 1920. American overseas shipping, however, differs sharply in this point from American railroads, that while, except for the Canadian lines, the railroads are immune

from direct international competition, American ships, with their higher wage scales and living conditions, are engaged in what is unquestionably the most intensely international competition in the world.

It should be emphasized that the international problem of the American merchant marine is not a complex problem, as is too often assumed. As a matter of fact, the whole proposition can be summed up in a few words: The American merchant marine in overseas trade used to be a vigorously protected National industry. As long as it was protected, even in part, by the preferential customs duties and tonnage taxes during the time of Washington, Hamilton, Jefferson, and Madison, and, afterward, by the mail subsidies of the Forties and Fifties, it flourished as incomparably the most efficient and prosperous shipping in existence. For sixty years, this American merchant marine, except the coastwise fleet, has been the one absolutely unprotected National industry, and for this reason, and no other, it had up to 1914 constantly declined.

Throughout the old years when American overseas shipping was protected, American ships conveyed on an average 80% of the imports and exports of the United States. On August 1st, 1914, American ships were conveying only about 10 per cent. All who are engaged in transportation will remember what happened in that war summer of 1914, when the German liners were withdrawn and "interned," and British, French, and Italian ships were summarily commandeered for the service of their Governments. Thanks to sixty years of neglect, there were then almost no American ships, and although the elevators and warehouses inland were full to overflowing, the railroad tracks for miles back from all the ports were choked with laden freight trains which could not be moved, because there were no ocean ships available. Commerce and agriculture for the time being were stricken with paralysis, at a cost of uncounted millions to American producers, and all for the lack of a real American merchant marine.

That experience shocked the country into a knowledge of the vital interdependence of agriculture, manufacturing, and shipping of its own. That was a lesson not easily forgotten. The President of the United States has addressed Congress and the country in a fervent plea for National consideration of the merchant marine to take the Government out of the shipping business and to make it possible for American ship-owners and seamen to live and prosper on the routes of ocean trade, and this specific policy should command the attention of all Americans interested in trade or transportation by land or by sea.

It is not a problem of creating a merchant marine. There is already a vast fleet in being. There is also a great corps of experienced and efficient officers and men, all the officers and a majority of the crews being American citizens. The question is one of aiding and protecting this fleet in being and its personnel so that the good tonnage of the Government may be transferred to private hands and operated by private initiative and enterprise under a National policy that will give American shipping the fair chance which the tariff has brought to manufacturing and to agriculture alike. It is the united

conviction of the practical men of the American ocean shipping industry that without the equivalent of that protection which other National interests have long enjoyed, the Government-owned, war-built tonnage will remain indefinitely in Government hands, subject to the inevitable disabilities of Government control and operation, and that private shipping, now nearly one-half of the ocean-going merchant marine—and with fair play destined to be the greater half of it—will again languish and decline, because it is forced unaided to fight the competition of a kind by which no other American industry is or has been confronted.

American ship owners, operators, and ship builders of ocean tonnage have unitedly gone on record to the Government of the United States as declaring that without a vigorous National policy, direct and indirect National aid, the ships of foreign Governments, backed under every flag by some form or degree of Government assistance, and manned under wage and living conditions which Americans will not accept, will again inevitably drive American ships from the ocean and destroy at once the delivery service of the ocean commerce and the resources of a National Naval Reserve. Profound is the significance to the American people of the statement with which Secretary Hughes opened the Disarmament Conference: "The importance of the merchant marine is in inverse ratio to the size of naval armaments!"

In the interest of peace, the Government is going to scrap a great part of the best of the American Navy. Every obligation of National security and prosperity, however, forbids that Americans should follow up the unexampled National sacrifice already made at Washington by the scrapping of their merchant fleet.

THE MERCHANT MARINE PROBLEM

By Emory R. Johnson,* Esq.

The merchant marine has been under consideration in the United States for more than two decades. A fairly comprehensive Shipping Act was passed in 1916, and four years later a still broader Merchant Marine Act was adopted by Congress. The passage of these two laws, however, has not removed the merchant marine question from public discussion, nor has the construction of a vast tonnage of shipping for war purposes determined the status and future prospects of the merchant marine.

The purpose in this paper is to consider briefly why a merchant marine under the American flag is really needed, and whether it will be necessary to give Government aid to shipping, in order to have an efficient and adequate merchant marine. In the preamble to the Merchant Marine Act of 1920, Congress declared "that it is necessary for the National defense and for the proper growth of its foreign and domestic commerce that the United States should have a merchant marine of the best equipped and most suitable types of vessels sufficient to carry the greater portion of its commerce and serve as a naval or military auxiliary in time of war or National emergency". In spite of this declaration of Congress, however, it is by no means certain that the man on the street or even the leaders of public opinion have thought out clearly why a merchant marine is necessary, and it is even more evident that there is a lack of unanimity of opinion as to whether Government aid will be necessary for the maintenance and adequate development of merchant shipping under the flag of the United States.

Ships are built and operated primarily to carry goods and passengers. If a merchant marine is really necessary, it must be so primarily because of its relation to domestic and foreign trade. Do the people of the United States need to be served by ships under the National flag, in order to develop their foreign trade successfully and largely in competition with other great maritime and trading nations?

The major share of the tonnage transported on the high seas is carried by chartered or "tramp" vessels. These day laborers in the world's ocean-carrying business constitute more than one-half of the large merchant tonnage under the British flag; and the thousands of "tramps" under the British, Norwegian, Dutch, American, and other flags may be chartered by shippers of any and all countries. The charges made for these charter services do not depend on the flag of the vessel. Charter rates are determined by world-wide competition. The big shipper in the United States who has a full vessel cargo can charter a ship without difficulty. Such being the case, why is it necessary that American vessels be available for charter?

Although the large shipper in any country can secure foreign vessels for chartered services, it is none the less a fact that, in general, the country having the largest merchant marine has an advantage in international trade. The existence of a large tonnage of shipping under the National flag is helpful

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to the establishment and development of foreign trade. This is due largely to the fact that ships are not only a necessary facility of foreign trade, but that they are one of several inter-related agencies that co-operate in making foreign trade successful and profitable. International commerce on a large scale, under the present conditions of competition, requires a world organization of industry, trade, and shipping. This organization begins with manufacturing and includes merchandising, international banking, marine insurance, ship brokerage, freight forwarding, and the construction, ownership, and operation of vessels. When this inter-related organization has been completely developed, as it has been by the people of Great Britain, foreign commerce can be carried on under the best of conditions. If shipping under the National flag is absent from the organization, as developed by the people of any country, the organization is less effective. Shipping under the National flag and an adequate tonnage of vessels of different types are, in fact, essential parts of the trade facilities and organization with which the people of the United States will be able to extend their commerce to all sections of the world.

The foregoing statement applies to ships of all kinds, those which operate under charters as well as those managed in line services. The relation of line services to trade development is, however, even more apparent than the relation of tramp tonnage to commercial expansion. Most of the shipments of importers and exporters and the major part of the business of freight forwarders require the services of regular steamship lines. Most shipments are in small lots, and the majority of manufacturers and traders seeking to build up foreign trade require services at regular and frequent intervals, such services as are rendered by established ocean lines. British traders, large and small, have long been able to place their exports in all foreign countries and to draw their supplies from the four quarters of the globe by means of the services of regular steamship lines operated under the British flag. If the people of the United States could be sure, within a reasonable time, of ocean-line services as numerous and efficient as those of the people of Great Britain, it is quite certain that the foreign commerce of the United States would be carried on and developed under more favorable conditions than those which now prevail.

One reason for a large merchant marine under the National flag seems to be quite clear to everybody, namely, that a large and well-balanced merchant marine is a necessary auxiliary to the Navy and in times of emergency may determine naval efficiency. In times of peace, the Navy may be limited to war vessels which may require the services of a comparatively small number of auxiliary vessels, such as colliers, tankers, supply ships, and transports. When war comes, however, these auxiliary craft must be greatly increased, and vessels of 14 to 20 knots in speed, manned by an efficient personnel, immediately become a necessity to the Navy. It is estimated that in time of an emergency the present Navy of the United States would require the services of 150 such merchant vessels.

As the result of the construction of ships during the World War, there is a large tonnage of vessels under the American flag, although the number of vessels of the size and speed required for naval auxiliaries is unfortunately

only a small part of the number that would be needed by the Navy in case of a sudden emergency. As will be pointed out subsequently the present merchant marine is not well balanced, either from the standpoint of commerce or from the point of view of the Navy.

The principal powers of the world are apparently about to start on a naval holiday of ten years' duration for the purpose of economizing in expenditures for ships and naval establishments. A naval holiday is much to be desired, but instead of diminishing, it will increase the importance, from the standpoint of National defense, of maintaining a well-balanced merchant marine under the National flag.

Both for commercial and naval reasons there should be a prosperous and vigorous shipbuilding industry in the United States. During the war there was a large increase in the number and output of private shipyards in the United States, but the world-wide depression in business and the temporary surplus of shipping, as related to the reduced volume of trade, have temporarily placed American shipyards in a precarious condition. Should this period of business depression be followed by a decline in the American merchant marine, or even by a serious postponement of the development of that marine, the shipyards in this country will suffer greatly, and probably many of them will be forced to go out of business. This would be a serious misfortune. One strong reason for adopting a policy that will assure the successful development of the merchant marine is the importance of keeping in vigorous existence the shipyards of the United States. These yards are necessary to National defense as well as to the healthy development of a merchant marine.

It is possible for an American citizen to secure a ship from a foreign builder, and register the vessel under the American flag for use in the foreign trade, but this fact does not make shipyards an unimportant part of the general organization required by the United States in building up a greater international trade. The commercial nations with which the United States must compete, Great Britain and Japan at the present time, and Germany in the future, are quite certain to have a large tonnage of shipping under their National flags. It should not be the aim of the United States to play a secondary or unimportant rôle on the high seas or in international commerce. It should rather be the policy of this country to establish, as far as practicable, all the conditions favorable to the shipping business and to the industries associated therewith.

There is another present and practical reason why it is desirable that American citizens and companies should find it profitable to own and operate vessels. The United States Government, through the Shipping Board, as a war measure, has brought into existence hundreds of vessels with a large aggregate tonnage. It is now clear to everybody that the Government cannot operate those vessels profitably. Congress has recognized this fact, and by the Merchant Marine Act of 1920 has directed the Shipping Board to sell the Government tonnage as soon as practicable and to retire at an early date from the operation of vessels.

The ships owned by the Government, that is, those that are commercially useful, ought to be purchased by Americans and operated in chartered and

line services in the foreign trade of the United States. The Shipping Board is offering its vessels for sale at competitive prices governed by world market conditions. It is to be hoped that these vessels will be bought by the people of the United States and operated successfully in foreign commerce. If the people of this country do not find it profitable to own and operate these vessels, presumably they must be sold to foreign buyers, and thus the people of the United States will be in the position of having constructed vessels that will contribute to the tonnage of shipping under foreign flags.

There is probably more unanimity of opinion as to the desirability of having a large merchant marine under the American flag than there is as to concrete measures to be taken to bring about the establishment, maintenance, and development of such a marine. From time to time before the World War, efforts were made to enact legislation providing Government aid for the merchant marine, but all these efforts failed with the exception of the enactment of the Mail Subvention Act of 1891—a very partial measure that has had comparatively little influence. To carry on the war against Germany, the United States constructed in Government yards, and purchased of private shipbuilders, a large tonnage of shipping, most of which the Government still owns. The major share of the tonnage owned by the Government, and also much of that owned by private individuals, is idle at present, because of the world-wide depression in business and international commerce. Again, an effort is to be made to bring about the adoption by the Federal Government of a policy of Government aid to shipping. Is such a policy necessary? Several facts indicate that Government aid in some form must be given in order to make profitable the ownership and operation of vessels under the American flag in the foreign trade. These reasons may be briefly stated as follows.

Almost without interruption, between the close of the Civil War and the beginning of the World War, the tonnage of shipping under the American flag in the foreign trade continued to decline. In 1914, less than 9% of the commerce of the United States, measured in value, was carried in ships under the American flag.

Even during 1921, at a time when private ship owners and the Government of the United States were in possession of a vast tonnage of shipping available for the service of those engaged in the foreign commerce of the United States, there was a decrease in the percentage of the total tonnage of the exports and imports of the United States carried in ships under the National flag.

The percentage of American shipping in actual service declined during 1921. The percentage of the total shipping that was idle increased, while, at the same time, the percentage of British shipping in actual employment increased. British ships were brought into use in increasing tonnage, while American ships were being tied up. This fact indicates clearly that it must have been more profitable to operate ships under the English flag than under the American flag in the foreign trade of the United States during 1921. The business depression could not have affected American shipping more than the shipping of Great Britain, Norway, or other foreign countries.

One of the most significant facts bearing on the question of Government aid to an American marine is that the present marine under the American flag is not well balanced. There is a large tonnage of slow cargo vessels most of which were built by the Government during and after the war. On the contrary, there are only a few vessels of large size and of a speed of 14 knots or more. The importance from the naval point of view of having a relatively large number of merchant vessels of 10 000 to 15 000 tons, gross measurement, and of speeds of more than 14 knots is very great. It is also necessary to have vessels of this type for the passenger and mail services and for the movement of high-class cargoes.

The American marine at the present time has an insufficient number of the following types of vessels:

(*a*).—Passenger and mail vessels of 15 000 gross tons and up, with speeds of 18 knots and more.

(*b*).—Passenger and cargo vessels of 10 000 to 15 000 gross tons, with speeds of 14 to 16 knots.

(*c*).—Fast cargo vessels of 9 000 dead weight tons and more, with speeds of 12 knots and more.

(*d*).—Refrigerator vessels of 8 000 to 12 000 gross tons, with speeds of 14 to 16 knots.

(*e*).—Vessels propelled by Diesel engines or by Diesel electric drive.

The significance of the foregoing statement as to the insufficiency of vessels of larger size and higher speed in the American merchant marine will be made quite clear by a brief comparison of the merchant fleets of the United States and Great Britain. In the British merchant marine, there are 632 sea-going vessels of from 2 000 to 3 000 tons gross, while in the United States marine there are 934 of these small vessels; whereas, the showing as to vessels of large size in the two fleets is directly the opposite. Great Britain has 190 merchant vessels of from 8 000 to 10 000 tons gross, while under the American flag there are only 85. In the British marine there are 138 vessels of from 10 000 to 15 000 tons gross, and in the American marine only 79. Under the British flag, there are 73 vessels of 15 000 tons gross and larger, while under the American flag there are only 14.

The deficiency in the American merchant marine as regards vessels of the higher types is indicated by the fact that in both the British and American fleets there are practically the same number of vessels capable of speeds of less than 12 knots, 2 556 under the British flag and 2 567 under the American flag. As regards vessels of a speed of 12 knots and more, there are 985 under the British flag and only 218 under the flag of the United States. Of vessels of speeds of 14 to 16 knots, there are 221 British and 53 American; of speeds of 16 to 18 knots, 92 British and 38 American; of a speed of 18 knots and more, 21 British and 7 American.

In another particular the merchant marine of the United States is unbalanced. The large refrigerator ship which makes possible the shipment of perishable commodities from any point of production to any part of the world, has come to play an important part in international trade. On June

30th, 1921, the British merchant marine contained 233 refrigerator vessels, with a gross tonnage of 2 000 000, while, at the same time, there were only 44 such vessels under the American flag, and their gross tonnage was only 220 000. In other words, the commerce of Great Britain was served by nearly ten times the tonnage of refrigerator vessels.

Argument is not necessary to prove that the efficiency of the American marine in competition with the merchant shipping of other countries depends on the construction and addition to the merchant fleet of the larger types of vessels of higher speeds. Moreover, the assistance which the merchant marine may render to the Navy in times of emergency depends on the number of ships of speeds of 14 knots and more and relatively large capacities.

In tonnage of vessels having oil-burning engines, the United States is in the lead, as a result of the equipment of a large part of the Government-built vessels with engines using oil for fuel. These are steam engines and mainly reciprocating engines. Experts believe that the most economical and efficient engines of the future will be those of the Diesel type. The fuel consumed is much less and the sailing radius is much greater. The initial cost is also higher. It is important that ship owners should be encouraged to convert some of their steamships into Diesel-engined vessels, and that those who order vessels in the future should be able to order an increasing percentage of new tonnage equipped with Diesel engines.

It can hardly be hoped that American citizens and American companies will find it profitable in the near future to construct, own, and operate vessels of large size and of relatively high speed without some form of Government aid, indirect or direct. The capital cost of such ships in American yards in the future will almost certainly be higher than in foreign yards. How much higher, it is not easy to determine.

Some significant facts in regard to the higher capital cost of American ships as compared with those built in foreign yards are fairly definite. Before the World War, the average cost of vessels was probably not less than 25% greater in American than in foreign yards. In many cases, the cost in America was 40% greater than in other countries. A careful estimate has recently been made as to the comparative costs of like vessels in American and foreign yards at the present time, and the conclusion has been reached that the cost in this country would be about 18 or 20% higher than in other countries. This differential, moreover, is not likely to decrease in the near future.

The man or the company that buys a ship in an American yard and operates it under the flag of the United States has two handicaps to overcome in competition with those who purchase ships in foreign yards and operate them under foreign flags. These handicaps are capital costs, or overhead, and operating expenses. Capital charges including interest, depreciation, and insurance amount to 17 or 17½% per annum on the investment in the ship. Thus, if a large freight vessel costs only $100 000 more when purchased in an American yard than when bought of a foreign builder, the owner has an annual excess capital cost of $17 000 or $17 500.

The operating expenses for ships under the American flag are unquestionably higher than for vessels under foreign flags. This is due mainly to differences in wages. For a standard freight vessel of 8 800 tons dead weight, the wages paid on the American ship during 1921 apparently averaged about $1 100 more than the wages paid on British ships. As compared with Norwegian vessels, the American wage handicap was even greater. There has been a readjustment of seamen's wages both in Great Britain and in the United States, and the difference in wages has been reduced, but it is still large, amounting to not less than $600 per month for a standard freight vessel. It also costs the American ship somewhat more for subsistence for its crew, and it is probable that the regulations of the United States Government as to inspections are more than those of foreign countries.

The United States Shipping Board is preparing a report which will set forth in detail the relative costs of American-built and foreign-built ships and the relative expenses of operating vessels under the American and foreign flags. The detailed results of this investigation are not yet available, but the figures will undoubtedly show that in the case of vessels of all classes, ordinary freight carriers, combination freight and passenger ships, and high-class passenger and mail vessels, the capital costs and the operating expenses are higher for the American than for the foreign ship owner and operator.

The inevitable conclusion is that if the American people are to have a large merchant marine in the future—a marine that can be operated profitably, and that will be progressively efficient—it will be necessary for the United States Government to give at least temporary aid to shipping.

WATER TRANSPORTATION IN ITS RELATION TO THE RAILWAYS

By Samuel O. Dunn,* Esq.

The competition that has existed, or that many think should exist, between carriers by rail and by water has been emphasized too much in the United States. The co-ordination of rail and water service of which there should be more, has been emphasized too little.

The policy of the Government in dealing with the rail and water carriers should have two purposes, both of great importance: First, to influence traffic to move by those routes over which the total cost incurred will be the least, considering the character of the service; and, second, to promote development of railways and shipping which will make them adequate to National needs in both peace and war. The speaker has referred to needs in war, because they should be given great consideration in shaping American marine policy. Wars are likely to come again in spite of all efforts to prevent them. The more that naval armaments are curtailed, the more vital will be the part played in any future wars by merchant vessels that can be converted into fighting ships. American business needs in time of peace and certain military needs in case of war combine to make a conclusive argument for a public policy that will promote the development of an American merchant marine as large as that of any other country.

The conditions are and will be such that the Government cannot, in some cases, carry out a policy that will both influence traffic to move by the cheapest route and at the same time foster the development of the merchant marine. There will be cases, for example, where it will be cheaper to ship American products overseas in foreign rather than in American vessels. The object should be to secure the cheapest transportation for American goods, that is consistent with the upbuilding of an adequate American merchant marine, as well as an adequate railway system.

The water carriers with which the railways have relations may be roughly divided into four classes.

The first class is composed of ocean-going vessels carrying goods between American ports and those of other countries. There is traffic which moves entirely by ship between the Pacific ports and Europe. There is traffic which moves entirely by ship between the Atlantic ports and the Orient. This traffic is handled partly by American, but mainly by foreign, ships. Directly or indirectly, the trans-continental railways compete for it. In the past they have made many rates intended to cause it to move partly by land and partly by water rather than entirely by water. For example, to capture traffic which otherwise would have moved by boat from the Atlantic seaboard, through the Suez Canal to the Orient, the railroads formerly made rates to the Pacific Coast, that were much lower on export than on domestic business and changed them as often as was necessary to get their share of the business. Public regulation required, however, that they should publish these rates. This

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rendered it impossible to make rapid changes in them, and the roads were also required not to go beyond certain limits in reducing them below their domestic rates. In consequence, a large part of this business was lost by the railroads.

Of course, however, the usual relationship between the railways and vessels in trans-oceanic service is that of connections rather than competitors. This is so true that there are some notable examples of trans-oceanic steamship lines being owned and operated by railway companies as through routes in connection with their railway lines. The Canadian Pacific operates vessels on both the Atlantic and the Pacific. The Great Northern under the management of the late James J. Hill, F. Am. Soc. C. E., established a steamship line on the Pacific to the Orient. The Southern Pacific owned the Pacific Mail Steamship Company and rendered service across the Pacific Ocean in connection with its rail lines. There have been other examples of the same kind, but, at present, practically none of the railways of the United States is operating boats owned by them in trans-oceanic service. The operation of trans-oceanic steamship lines by railway companies has benefited Canada, and probably would benefit the United States, but the American policy of regulation of railways discourages it. Although the American policy of regulation of railways discourages them from owning and operating trans-oceanic steamship lines, an attempt is now being made to force them to favor American vessels in handling export and import freight. The two policies seem inconsistent.

The Government, undoubtedly, should use its influence to bring about every form of co-ordination between the service and rate-making of the railways and the merchant marine of the United States. Neither morally nor economically, however, would it be justified in requiring that the railways establish rates and service in connection with steamship lines, which would cause losses to the railways. It is no part of the duty of the railways, directly or indirectly, to subsidize the steamship lines any more than it is a part of the duty of the steamship lines to subsidize the railways. The American merchant marine should be built up for the benefit of the entire nation and, therefore, insofar as it must be subsidized the entire burden of the subsidies should be borne by the public through the Government. If the railways are to be required to favor American ships in the matter of contract relations, interchange of traffic, or otherwise, it should be done in a way which will be helpful to the merchant marine without being injurious to the railways which have just as good a claim to fair and helpful treatment from the Government as the merchant marine.

The laws give American ships a monopoly of the coastwise trade. Ocean transportation is so much cheaper than rail transportation, where the distances by rail and water are anywhere near the same, that it is impossible for the railways to compete with it. Even where the distance by water is much greater than by rail, as it is by the Panama Canal route between the Atlantic and Pacific Coasts, the railways can hardly hope to compete for traffic that originates on or near one coast and is destined for points on or near the other coast. The conditions are different when traffic originates in, or is

destined to, points in the interior and, therefore, must be handled either partly by rail and partly by water, or entirely by rail, and this is true of most of the traffic for which the railways compete with the Panama Canal route.

The question of what relations should be established between the railways and coastwise vessels using the Panama Canal is still the subject of much controversy. The railways are prohibited from owning vessels operating through the Canal. The vessels are now required to pay tolls, but in spite of this are taking an increasing amount of the competitive traffic away from the railways. Attempts are being made to secure the abolition of the Canal tolls, which would give the vessels an increased advantage. Meanwhile, the railways are asking the Interstate Commerce Commission to allow them to reduce their rates to the Pacific Coast without making corresponding reductions in their rates to interior points, in order that they may meet the competition of the vessels to points on the coast without, at the same time, reducing their earnings on traffic going to the interior.

Here, the problem of establishing proper relations between the railways and water carriers is found in its most complex and difficult form. It cannot be solved correctly merely by considering what is to the interest of the steamships or the railway lines. It can be solved correctly only by considering what is to the interest of the nation as a whole. Americans should spare no reasonable effort to build up their merchant marine. That is in the National interest. At the same time, they should not in order to build up the merchant marine, adopt a policy which will tend to tear down those railways which seek to share in this traffic.

The competition which presents the problem is, it should be noted, not between means of transportation owned on the one side by Americans and on the other side by foreigners. It is between two kinds of transportation agencies both of which have been provided by American capital and both of which are equally entitled to be fostered, or at least not injured, by the American Government. The speaker feels so strongly in favor of the upbuilding of the American merchant marine that if it was necessary to subsidize it to enable it to compete successfully with the merchant marines of other countries for American commerce and that of the remainder of the world, he is in favor of subsidizing it from the public treasury as much as may be necessary. Other nations do this, and Americans cannot safely refrain from doing so while they do it.

However, when the question presented is one arising from competition between American railways and American ships, the latter operating through a canal built for taxes paid by the entire American people, it seems to the speaker that such influence as the Government may exert should be used to cause the traffic to move by the cheapest route, differences of service being considered. What, however, is the real measure of the relative cheapness of rival routes? It is not merely the rates actually charged the shipper at any given time. If, for example, the coastwise vessels were exempted from tolls and reduced their rates to the shipper correspondingly, this would not mean that the cost of transportation by the Canal route had been reduced. It would

simply mean that the public was paying in taxes part of the cost which the shipper before had paid in the rates. The true cost of transportation by any route, rail or water, is the rate for which the carrier rendering the service can reasonably afford to haul the shipment without receiving any contribution toward payment of the cost of hauling it, except the rate charged. This is an abstract statement, but the speaker will use the very question under consideration to illustrate concretely what he means by it. The railways to the Pacific Coast have no source of income but their rates which, as a whole, must cover all their outgo. They can better afford to reduce their rates on this trans-continental traffic below the average rates they obtain on all their traffic and thereby get some of this trans-continental traffic than go without it entirely. There is, however, a limit below which the railways could not reduce their rates on trans-continental traffic without making the earnings from it less than the out-of-pocket expense incurred in handling it. The railways can be trusted, because of their own self-interest, to let the traffic go entirely rather than reduce the rates to a point where they will be actually unremunerative. The speaker maintains that if the coastwise vessels cannot pay tolls which will contribute reasonably toward the interest, maintenance, and operating charges of the Canal, and at the same time operate at a profit on rates low enough to meet the rates on trans-continental traffic that the railways find they can better afford to make than give up the business, then the rail route is actually cheaper than the water route and the business ought to move over it.

As a matter of fact, there is no doubt in the speaker's mind that the coastwise vessels always will be able to pay substantial tolls and at the same time carry most of the trans-continental traffic. To exempt them from the tolls and at the same time prohibit the railways from making lower rates to the coast than to the interior would be to drive the railways entirely out of the trans-continental business and force them to get all their earnings from traffic for which they do not compete with the steamship lines.

The relations of the railways to water carriers on the Great Lakes are like their relations to most coastwise carriers. The cost of transportation on the Great Lakes is so much less than it is by rail for traffic that can be handled entirely or almost entirely by water that the railways cannot compete for this traffic. The water carriers have a virtual monopoly of the transportation of coal to the Northwest from the Pennsylvania and West Virginia fields and of ore from the Northwest, the railways simply serving as feeders to them. The lakes are closed to navigation in the winter, however. This gives the railways, which can operate the entire year, an advantage in respect to the relative costs of rendering service that they otherwise would not have. Furthermore, the traffic between the North Atlantic seaboard and Chicago must be transferred *en route* if it moves part way over the Great Lakes. Finally, the railways are better able than most of the lake carriers to give prompt service and to maintain agencies throughout the country for the solicitation of business. Consequently, the all-rail routes between Chicago and the Atlantic seaboard are able to compete for most kinds of business with

the rail and lake routes. Formerly, several railways owned and operated boat lines on the Great Lakes, which formed direct through routes in connection with the Eastern rail lines. Legislation intended to increase competition between the lake lines and the railways prohibited this railway ownership of boat lines except when authorized by the Interstate Commerce Commission. The result was that most of the railways were forced to sell their boat lines and the service on the lakes was seriously reduced. The legislation, instead of conferring a public benefit, accomplished a public injury, as such legislation not infrequently does.

The only remaining waterways with which the railways have any relations are canals and improved rivers. The Government has spent large sums in trying to develop canals and rivers which could compete with the railways. It has passed legislation to prevent the railways from reducing rates solely to kill water competition and later increasing them; but while the traffic of the railways has continued to increase, the traffic of most canals and rivers has continued to dwindle. Probably one of the main reasons for this is that the Government has wasted its energies and money by spending them on many projects instead of by concentrating on a few of the most promising. The Government has been making an experiment with inland water service which is of real interest and importance. This is on the Mississippi and Warrior Rivers. It has provided at public expense improved towboats and barges and has built new water terminals at several points. On their face, the results thus far obtained are not highly encouraging. In the year ended June 30th, 1921, the operating deficit incurred on the Mississippi River section was $503 000 and on the Warrior River section $480 000. On the Mississippi River the average revenue from each ton transported was $4.39, whereas the average operating expense for each ton transported was $6.56. On the Warrior River section the average revenue for each ton transported was $1.86, and the average operating expense for each ton was $4.11. The report shows, however, that in the three months, April, May, and June, an actual net profit of almost $23 000 was earned on the Mississippi River section.

Although the speaker has stated that the results thus far obtained from this ambitious project are far from satisfactory, he believes that the Government should be encouraged to continue the experiment and that in this and all other cases the railways should freely co-operate with the water carriers in the interchange of traffic and refrain from making rates intended mainly to drive them out of business. There has been much discussion in the United States of the relative costs and advantages of rail and water transportation. This discussion insofar as it has related to rail transportation *versus* canal and river transportation has been peculiarly barren, because never has canal and river transportation been tested under such conditions with respect to depth of waterways and character of towboats, barges, and terminals as to enable one to judge intelligently of the relative costs of modern inland water transportation and modern rail transportation under American conditions. Personally, the speaker does not believe that inland water carriers, except on the Great Lakes, can ever compete successfully with the railways of the United

States if the character of the service as well as all the costs incurred in rendering the service are considered. In the total cost of water transportation is included, not only the cost of maintaining and operating the boats, but also the cost of maintaining the waterway and interest on the investment made in improving it. No comparison of the cost of water transportation with the cost of rail transportation is fair or reasonable, which does not include these items. More real evidence is needed regarding the actual cost of rail and inland water transportation from which to draw final conclusions.

DISCUSSION

GARDNER S. WILLIAMS,* M. AM. SOC. C. E.—Considerable has been said, as to the relations between the carrying capacity of the American sea-going merchant marine and the exports and imports. There must be some legitimate ratio between the tonnage of exports and imports which go from and come to the United States and the proportion which its marine is entitled to carry, assuming that each nation in the world has its share. The world is gradually coming to the point when no one nation will be permitted to take all the profits of any particular line of business. It is obvious that the United States cannot expect to carry all its exports and its imports, because if it did, there would be some nations that would have nothing to carry. If the United States has the right to carry its exports, then every nation which would export to this country has a right to carry its exports. A merchant marine cannot be operated which has cargoes only one way, so a portion of the imports must be carried as well as the exports. The statisticians should determine just what proportion of the imports and exports of the United States it is rationally, legitimately, and fairly entitled to carry, in justice to its neighbors in this world, and how far that proportion is from the amount carried. If every nation were a carrier, the answer would be one-half of each, but some nations are not carriers, hence there is a problem to be solved.

On the Great Lakes, it has been found that a certain speed of vessel was the most economical for the service required there; or, rather, that, for particular services, vessels of certain speeds are the most economical. That same condition must exist on the ocean. Probably that has been investigated, but the speaker does not know where to find the figures. In regard to the matter of subsidy, it seems that a subsidy should only be used where the conditions of the service require that the constructors or the maintainers of that service should depart from that which is most economically profitable; in other words, if it is necessary that ships of a higher speed than is economically profitable are introduced into the service, then the Government should countribute to that; but if it is not necessary that the ships should be built for any more than the most economical operation, then it seems that it is questionable whether a subsidy should be granted. Many believe that if the Government will remove some of its restrictions, private industry will overcome the remainder of the difficulties. Conditions exist in this country, that do not appear in all the countries abroad. It is unfortunate perhaps that the American wants to live on such a high plane or, perhaps, it is a good thing that he does; but there are times when it militates against the prosperity of the country as a whole, if he insists on having things which are not necessary. These, however, are problems that probably Government cannot handle; but if the United States cannot maintain a merchant marine without bringing a greater burden on itself than the benefit which the marine will bring, true statesmanship would decide that normal industry must take its course.

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J. E. Willoughby,* M. Am. Soc. C. E.—The speaker agrees with the views expressed that the Federal Government should render some aid to the trans-oceanic shipping, but such aid should not be rendered to coastwise vessels, because it will be subsidizing those vessels to the injury of the coast-line railroads.

The speaker has the impression that there are, on the statute books, various laws with respect to wages and working conditions that make for waste in marine service, and it is his opinion that the attention of ship owners might properly be directed to the repeal of those laws, as well as to subsidies, which are necessary in order to make the merchant marine a success. The railroad with which the speaker is connected has long worked in co-operation with certain coast-line vessels as a distributor to interior points of traffic from the North Atlantic ports to those of the South Atlantic. In the past, a success has been made of that distribution, and it will be continued. Recently, there has been established, in compliance with the orders of the Interstate Commerce Commission, certain rates that affect traffic between the ports mentioned, and these rates will cause the abandonment by the railroads of part of the traffic between those ports (other than such as will be distributed to interior points), and establish the carrying of that traffic by coastwise vessels as the cheapest routing. Transportation under the new rates between the port cities of the South and North Atlantic is so much cheaper by water than by rail, that the railroads are abandoning the carriage of certain commerce.

John C. Trautwine, Jr.,† M. Am. Soc. C. E.—The papers on "Water Transportation", indicate that a conflict of interests exists, not only between water and rail transportation, but between these two combined, or, briefly, "capital" and "labor", to say nothing of the public interest as an apparently negligible third.

For instance, Dr. Johnson states that American ocean shippers are at a disadvantage, as compared with those of Europe, by reason of the higher wages prevailing here; and although it is the fashion to deprecate governmental interference with "business", Government is looked to, to come to the aid of the impoverished American shipper, as against the American laborer.

As long as economy remains based on the obsolescent profit system, the business structure is necessarily a pyramid standing on its apex; and it is the delicate task of the Government to prevent it from toppling over, in one direction or the other, and crushing this or that "interest" in its fall.

Augustus Smith,‡ M. Am. Soc. C. E.—The remarks by Mr. Williams and Mr. Trautwine have made the speaker realize that his business, which is fabricating structural steel, is related in some way with this matter of water transportation.

The speaker has in his library a book, entitled "Tales of an Old Sea Port." The stories are of famous American voyages and ships that sailed from Bristol, R. I., and carried the American flag to all the "seven seas." An

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† Philadelphia, Pa.
‡ Pres., Bergen Point Iron Works, Bayonne, N. J.

outstanding feature is that, in those days, the crews from captain to cabin boy, participated in the profits of the voyage. Much the same economic arrangement exists to-day in the fishing industry at Gloucester and elsewhere.

The speaker offers the thought that it might be possible to stimulate American shipping anew if this old co-operative basis was again adopted, having legislation amended, if necessary, so that when owners and crews so choose, they might share in the profits of a voyage.

Thus, a method of computation might be to estimate a vessel as worth so many thousands of dollars, a seaman worth, say, $15 000, a captain, say, $30 000, and so on, placing a capital value on every one of the ship's company. When the ship returned to its home port, the profits would be divided according to the "invested capital". On such a basis, if a vessel earned fuel, food for the crew on the voyage, and the necessary repairs and maintenance, there would be no "loss", and men in the business of fabricating structural steel on shore would not have to be called on to make good any deficit. Such voyages would appeal to the spirit of adventure in all young men and would attract good American blood and might well yield handsome profits.

C. C. Vermeule,* M. Am. Soc. C. E.—At the foundation of all sound engineering lies economics, and by his training and experience, the engineer is fitted to lead the way to a rational economic policy on which regulation of transportation and expenditures of public money for any purpose should be based. Speaking from the standpoint of the great industries of the country, the time has come when strict attention must be paid to conserving public funds and restoring to private enterprise a reasonable freedom of action. A serious difficulty arises when Government undertakings compete with private enterprise in the field of transportation, or other business, for, at once, the normal economic balance is upset. This is mainly because public money is not used, and there is not sufficient incentive to its use, with the same strict insistence on proper financial returns that obtains when private capital is utilized. The latter must earn dividends or be driven to bankruptcy, whereas the Government enterprise can always, but improvidently, dip more deeply into capital account to make up a deficit in earnings.

No invidious application to the operations of the Shipping Board is intended. The Board inherited an economically monstrous condition at the end of the World War. It should be considered, however, what would have been the course had a business man found himself confronted with the same state of affairs in a private enterprise. As a good business man, he would unquestionably have made up his inevitable losses as promptly as possible and started afresh on a sound, economic basis. Is there any sufficient reason why the same course should not have been followed by the Government? If industries and agriculture are to prosper, they must not be forced to maintain, through a false National pride, a merchant fleet which cannot furnish them transportation at rates as low as their competitors can obtain. Nor can this be a fictitious rate, with resulting deficits to go into the tax bill. Nothing but really economical transportation will permanently help

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the industries. Competition has now become international, and if business is to thrive, it must be able to command the markets of the world.

It recently became necessary, in connection with industrial operations, to make some forecast of the future level of wages and prices, a question which so often presents itself to the engineer in his work. It was determined that one of the most important factors which will influence that level, is to be public expenditure. The effect of reckless use of public money by National, State, and municipal governments, is that the average worker in the United States must produce $200 a year for Government expenditure. There are 42 000 000 workers engaged in gainful occupations in the United States, and it is well known to students of economics that their average earnings in 1915, or just before this country went into the war, only amounted to $680 yearly. An investigation covering sixty years reveals the fact that at no time has it been found possible to collect directly or indirectly from the worker more than one-seventh of his earnings for Government use. Consequently, on the 1920 basis of public expenditure, he must earn not less than $1 400 annually, or more than twice as much as before the war. Many workers are being paid this much, but are they earning it? If not, it is coming out of the stored up National wealth and the country is growing poorer. Some economy has recently been effected by the National Government, but State and municipal expenditures still tend to increase, and the blame for this rests more often with the people, who advocate every conceivable project requiring public money, than with the office holder. Due mainly to the scale of public expenditure, it seems impossible that wages will drop to a lower level than 45% over pre-war figures, which means that prices of all commodities will not fall permanently below the same level, as wages will always be a controlling factor in costs.

Unfortunately, a large number of people believe that taxes can be collected from the rich alone. It is still more remarkable, notwithstanding it has become an established principle that public utility rates are determined by cost, that the people believe that these enterprises can be taxed without the bill coming back to them in rates. It is most important that engineers, who know better, shall do all in their power to correct these errors, for as long as they prevail, the nation will never come to a sound, economic basis. When every one realizes that every dollar of public money must be contributed from private funds, the public will insist that such money shall be used with the same care and shall produce the same financial and economic result that is insisted on when private capital is utilized in enterprise. It may be easily shown that if the rich or the provident alone were called on to pay all the taxes, their liquid assets would be exhausted at the end of two years, and when that point was reached, the remainder of their assets, with most of the National wealth, would become worthless for lack of purchasers. Again, taxes cannot come out of the industrial or transportation enterprises, for there is hardly one of them that could survive more than three years if it could not recover its taxes through increasing the prices of its product. Many of the great transportation companies are paying in taxes from 9 to 10 cents out of every dollar which they collect in rates, and it should be self-evident that both these and

the public utility companies must increase their charges to the public to provide the money to pay these taxes.

It is useless for the Government to build fine highways or other public works, or to conduct a shipping industry or other transportation enterprises, if it thereby takes from the worker so much of his earnings that his purchasing power is greatly reduced, or if it destroys manufacturing and agricultural interests through excessive taxation. Consider for a moment the effect on the smaller industries, for it will illustrate the harmful results. Assume a concern beginning with $100 000 capital, and manufacturing, perhaps under patents, a useful product which enables, in a given year, a profit of 100% to be realized, and some of them are fortunate enough to do so. During the next year, their growing business will certainly require twice as much capital, but their profits are largely locked up in additions to plant and book accounts. Nevertheless, as conditions have been recently, the Federal and local governments call on them for perhaps $50 000 in taxes, which must be paid out of liquid assets. Bankers do not care to loan them money to pay taxes, and, consequently, these young concerns are deprived of capital which they need to provide for their growing business, and they cannot employ labor, expand, and produce. It is by such healthy young enterprises that the best business of the country has been built up, and to prohibit their activities, causes a serious loss to the wealth of the nation at large. These are the enterprises which will furnish freight to the railroads and ships of the future, and which must be able to obtain their ocean rates on a basis that will enable them to compete with the other industrial nations of the world.

In this connection, the effect of unwise Government regulation of railroads upon the industrial situation may also be considered. No better illustration is known than the story of freight on carload lots of steam coal in New England, for the cost of coal is a most important factor in the future of industries. From 1895 to 1916, the average level of wages and prices was increasing at the rate of 10% compounded every 5 years, but through ill-advised Government regulation, freight rates had not been allowed to advance and were slightly lower in 1916 than they were in 1900. New England industries had accommodated themselves to these fictitious low rates, and much money had been invested thereon. It was found, when the Government faced war, how short-sighted this crippling of the railroads had been, and the rate from the Cumberland District to points near Boston was suddenly jumped from $2.95 to $5.28 per ton. This sudden advance has been most disturbing to industry, and, particularly, since the level of prices has fallen and business has come again to a strictly competitive basis. Because of the great economies effected in railroad transportation, it would not have been necessary to increase the rates as rapidly as the average of wages and prices. Had this been done, it would have brought the rate on coal to $4.20 in 1916 instead of $2.95. It is perfectly apparent, however, that the rate in 1915 should have been 25% higher than it was, or, say, $3.75 per ton. Had this been permitted, industry would have accommodated itself thereto, because the increase would have been gradual and proportional all over the United States, and could have been easily absorbed. It would seem best to come back to the old principle of charging

what the traffic will bear, for to destroy all industries remote from the coal fields is unthinkable. It is necessary and just, however, that the railroads, in their present condition, shall be allowed a somewhat higher rate than $3.75, in order that they may make up for past deficiencies, due to unwise Governmental interference.

Past experience shows, therefore, that it is dangerous to the business of the nation when the Government enters competitively therein; there is also a limit to the taxes which can be imposed on industry to furnish money for unwise undertakings. Although the principle of public regulation of utilities has been accepted as correct, no way has yet been found to apply it wisely, and as now practiced it is also a menace to the whole business of the country. Last, but not least, the merits of all proposals to expend large amounts of public money should be most carefully examined and from this examination should be excluded all mere pride of achievement of ends that will not produce a direct and certain profit to the average worker whose earnings the Government is investing. Whether such expenditures of public funds would be entered into with private capital, by reasonably conservative business men must be considered seriously. If not, there should be sound reasons to justify them, before they are undertaken. Everybody knows of many things they would like, but for which they cannot pay; the nation is now in the same condition.

The trained minds of the Engineering Profession are needed to concentrate patriotically and unselfishly on these great questions, and it is their duty as good citizens to contribute all they can to the establishment of a wise, conservative, economic policy, for as stated previously, as the basis of true engineering, of real accomplishment, and of prosperity in the profession, we must have sound economics, no matter whether the funds to be invested are public or private.

B. F. Groat,* M. Am. Soc. C. E.—The speaker will mention what he believes is nearest the hearts of those interested in water transportation, namely, the question of a subsidy. As a general rule, the speaker is constitutionally opposed to subsidies, but it seems that an American merchant marine is an exception to this rule. The United States has to compete with the remainder of the world, and if the remainder of the world can bring in commodities cheaper than this country, when measured in dollars and cents, which is not always a safe measure, then he is in favor of the subsidy.

There are many ways of granting a subsidy. An example of the price of water power will illustrate. The water-power policy in this country seems to demand that such power be sold at a less price than steam power of the same character. In the case of steam power, a certain supply may be had from a given plant, which is not the case with water power. River flow is variable, and one cannot afford, in all cases, to spend money enough to regulate the river completely. Therefore, there are two kinds of water power, continuous and variable. The continuous water power is like steam power, it is always available. The variable kind may be divided into two parts, or

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kinds, one of which will be called dependable seasonal power and the other mere surplus power which comes in varying quantities and for variable periods, sometimes lasting for months, or even years, only to vanish as unexpectedly as it appeared.

In the case of continuous power, there should not be any difference between the price of water power and that of steam power, assuming an open market, just as there should be no difference in the price of commodities from one section of the country as compared with the price of similar commodities from another section, because one farmer can produce his crops more cheaply than another. There is a market for such things. In the case of water power, however, the policy seems to be to compel the sale of water power for less than the market price of steam power.

What is the result? For example, take a large manufacturing establishment which can obtain a supply of water power at a price lower than the market price of steam power. That establishment immediately has the advantage a subsidy would give. Such subsidies are granted in such a manner that they are frequently unrecognized as such by the grantor, and thus some great industries have been subsidized. If one examines the development of water power at Niagara Falls, he will appreciate this fact. The manufacturers get the power at comparatively low rates. The manufacturer has, in effect, a subsidy, and it is granted substantially in the manner described.

With a merchant marine, it is not only a question of economic advantage to maintain it, but, to a certain extent, it is also one of patriotic pride. One may say that patriotic pride is not an economic force, but the speaker is convinced that it is. Who will belittle the economic value of having the American flag flying in every harbor of the world?

Only one other point will be discussed, and this was touched on by Mr. Williams and others, that is, the best speed of a ship. The speaker does not believe there is any such thing, that is, when one takes into account all the variable factors at one period and under one state of economic and mechanical conditions, there will be one speed, and under changed conditions there will be a different speed. Here, again, the same thing is found in water power. There is said to be a best speed for a water-wheel, and yet, when one tries to apply that principle practically, he will find it a difficult problem. The best speed varies from day to day, according to mechanical and hydraulic as well as other conditions; and the speaker thinks the same thing must be true of ship speeds.

It seems not unlikely that ships should have different types of propellers to meet varying conditions, especially the economic conditions. The speaker would not dare to say that economic conditions will not arise requiring that sort of thing, although he is aware of the nature of the difficulties to be overcome in providing suitable marine power plants for such purposes.

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RAILROAD TRANSPORTATION*

A SYMPOSIUM

By Messrs. Howard Elliott, W. N. Doak, and F. A. Molitor.

With Discussion by George W. Simmons, Esq.

* Presented at the meeting of January 19th, 1922.

RAILROAD TRANSPORTATION

By Howard Elliott,* Affiliate, Am. Soc. C. E.

It was the speaker's good fortune, from 1903 to 1913, to be President of the Northern Pacific Railway Company.

In that decade the owners and directors spent more than $100 000 000 in improving the property in a physical way, and a large amount of work was done in making the Northern Pacific a transportation machine that could produce a maximum of transportation units with a minimum expenditure of physical energy. This was for the ultimate purpose of giving to the growing communities between Lake Superior and the Pacific Ocean, good transportation at the lowest practicable rates consistent with a fair return on the fair value of the plant.

In that interesting work, the speaker had the assistance, as Chief Engineer, first of E. J. Pearson, M. Am. Soc. C. E., resourceful in design and experienced in the use of modern railroad facilities, and who is now President of the New York, New Haven, and Hartford Railroad Company, and, later, of W. L. Darling, M. Am. Soc. C. E., a past master of railroad engineering problems.

The Stockwood Fill on the Northern Pacific.

One interesting piece of work was the revision of the line and grade on the Northern Pacific in order that heavy tonnage trains could be moved from the Red River Valley over the Divide into the Mississippi Valley. An east-bound grade of 0.30% was decided on in place of the existing maximum of 1.5 per cent. To accomplish this an embankment was required 7.3 miles long, having a maximum height of 49 ft., and a total yardage of 5 155 694, of which 4 067 844 cu. yd. are above, and 1 087 850 cu. yd. are below, the plane of the original ground line.

After the work was started, it was discovered that the bed of a prehistoric lake was beneath part of the heavy fill, the old lake bed being filled with a sedimentary deposit of sand, saturated with water, for about 20 ft., underlaid by soft blue and yellow clay. The result was that the heavy fill broke through the surface and the continuity of the 0.30% grade line was broken, forcing the introduction of a short piece of 1% grade, which, however, does not limit east-bound train tonnage, except in bad weather, when helping engines have to be used. This work was begun in March, 1906, and completed in December, 1909. For ten years there has been little further subsidence, and it is possible that in due time the entire embankment can be brought up to the original 0.30% grade.

The Line Between Vancouver, Wash., and Portland, Ore.

Another interesting work with which it was the speaker's privilege to be associated, was the building of the Spokane, Portland and Seattle Railway, which has a 0.40% grade between Spokane, Wash., and the Snake River, and a 0.20% grade along the Snake and Columbia Rivers to Portland, thus making

* Chairman, Executive Committee, N. P. Ry., New York City.

a fine line for the development of the Inland Empire and the Columbia River Basin, capable of being operated cheaply when any substantial tonnage is moving. The entire line along the Columbia and Snake Rivers and from the Snake River to Spokane, is most interesting from an engineering (as well as from a commercial) standpoint, and has much heavy rock work through and around basaltic cliffs and many heavy fills across "coulees".

The Spokane, Portland and Seattle Railway is owned jointly by the Northern Pacific Railway Company and the Great Northern Railway Company and uses certain parts of the Northern Pacific, the bridge over the Columbia River, at Pasco, Wash., and the entrance to Portland on the west bank of the Willamette River; as well as the old Northern Pacific main line to Goble, Ore., where connection is made with the Astoria and Columbia River Railroad, purchased by the Spokane, Portland and Seattle, thus forming a low-grade line between Spokane and Astoria, Ore., at the mouth of the Columbia. There is also a branch, 156 miles long, to Central Oregon. The Northern Pacific Company, with its main north and south line between Portland and Tacoma and Seattle, Wash., joined the Spokane, Portland and Seattle (of which it is a one-half owner) in creating a remarkable piece of railroad from the north side of the Columbia River at Vancouver, Wash., to the west bank of the Willamette River at Willbridge, in North Portland, Ore.

This line is made up of the bridge over the Columbia River, Hayden Island, and Oregon Slough, 6 467 ft. of bridge, of which 2 194 ft. are girder spans over Hayden Island and 4 273 ft. truss spans over the two channels, with draw spans of 467 ft. and 333 ft., respectively. The maximum depth of the foundation below low water is 80 ft. and below the base of rail 126 ft. About 3 miles of road have been constructed across the St. John Peninsula, in Oregon, in order to reach the Willamette River which has a heavy bluff on the east side. In order to maintain the grade line, this bluff had to be cut and 2 357 000 cu. yd. of material were taken out. Then, the Willamette River Bridge, 1 767 ft. long, with a draw span, 521 ft. long, was built, the largest double-track draw span in the world at that time. The maximum depth of the foundation below low water is 86 ft. and below base of rail 148 ft.

There were used in the construction of the two bridges:

Masonry and concrete	79 012 cu. yd.
Steel superstructure	22 175 tons
Timber in foundation	6 927 000 ft. B. M.
Piling	296 860 lin. ft.

and the two bridges cost $4 160 000, a sum far less than they would cost to-day.

Work was begun on the foundations for the Columbia River Bridge on February 8th, 1906, and the superstructure was completed on February 26th, 1908. Work on the Willamette River Bridge was begun on August 11th, 1906, and the first train between Portland and Vancouver was moved on November 5th, 1908, and regular passenger service began on November 17th, 1908. The bridges and the connecting tracks are double track, and are now being used by

the Northern Pacific, the Spokane, Portland and Seattle, the Great Northern, and the Oregon-Washington Railroad and Navigation Company of the Union Pacific System. Mr. Darling was Chief Engineer of the Northern Pacific and the Spokane, Portland, and Seattle at that time, and Ralph Modjeski, M. Am. Soc. C. E., was in charge of these two great bridges.

As time goes on, this Spokane, Portland, and Seattle line to Astoria, and this entrance to Portland, will play a more and more important part in the development, commercially and socially, of that interesting part of the Northwest between Portland and Bend, Ore., on the south, Astoria on the west, Vancouver, B. C., Canada, on the north, and east into the Inland Empire and the Columbia and Snake River Basins.

Work on the New Haven.

On the New York, New Haven, and Hartford Railroad, it became absolutely necessary to add to the capacity of that property if it was to serve the large tributary population. Mr. Pearson was asked to associate himself with the property and, with him, plans were developed for the work that is gradually being finished under his direction. This addition increases the ability of the property to handle its heavy freight tonnage and numerous passenger trains and reduces the cost per unit of transportation produced. Two important pieces of work are the large freight yards at New Haven, Conn., and Providence, R. I.

Importance of the Engineer to Transportation.

Some of these things are mentioned to emphasize the great importance of the engineer in the transportation problem of the United States, not only in creating the original transportation machine, but in perfecting it so that there will be the maximum output at minimum cost in these days of complications, high wages, and other costs, affecting the administration of any industrial plant. Also, to show how much benefit the speaker's education as an engineer and his association with engineers has been in his work in the railroad world.

Railroad Transportation.

The speaker has been asked to say something about "Railroad Transportation", which is so vital to the present and future development of the United States and its people, agriculturally, industrially, financially, and socially. Without easy means of communication, it is obvious that the highest development of our wonderful resources and the consequent improvement of living conditions cannot be obtained.

Record of Half a Century.

Let us examine briefly the development of the railroad system in a half century—a development made possible by men of imagination, vision, and courage; by financiers and investors who took chances; and last, but not least, by engineers who have been able to carry into effect the plans and purposes of the other two groups.

TABLE 1.—Miles of Steam Roads—Single-Track—Owned.

	United States.	Eastern District.	Southern District.	Western District.
1870 (June 30th)	53 399	27 068	11 888	14 443
1920 (December 31st)	253 708	61 296	51 274	141 138
Percentage of increase	375	126	331	877
Population:				
1870 (June 1st)	38 558 371	21 693 654	7 899 435	8 965 282
1920 (January 1st)	105 710 620	51 565 264	18 356 714	35 788 642
Percentage of increase	174	138	132	299

TABLE 2.—Urban* and Rural Population.

Year.	Urban.	Rural.	Percentage of urban.
1880	14 358 167	35 797 616	28.6
1890	22 298 359	40 649 355	35.4
1900	30 380 433	45 614 142	40.0
1910	42 166 120	49 806 146	45.8
1920	54 304 603	51 406 017	51.4

* Urban population comprises that population residing in cities and other incorporated places of 2 500 inhabitants or more, and in unincorporated towns of 2 500 or more in Massachusetts, New Hampshire, and Rhode Island.

It is interesting to note that the railroad mileage has increased twice as fast as the population (mileage 375%; population 174%). In the great empire west of the Mississippi, mileage has increased 877%, and the population 299 per cent. This indicates clearly that the men of courage and vision, the men with ability to raise money, the engineers with ability to design and construct have done a great work in providing the present marvellous transportation machine that is so essential if the 107 000 000 people who are crowding too much into cities and away from the country, are to be sheltered, fed, clothed, kept warm, and safeguarded in their various occupations and in their homes. These men ought to receive commendation and thanks for what they have done and made possible for the country, and the great transportation machine ought to be allowed a living and be encouraged to expand.

Unfortunately, for the future growth of the United States, there is too great a tendency on the part of the public and the lawmakers to see only the minor mistakes incident to the creation and operation of this great transportation engine, which is performing such prodigious work for all (hampered by many laws). They fail to appreciate the whole wonderful work and what the Nation would be without it. They hold the "penny" of failure so close to their eyes that they do not always see the "twenty-dollar gold piece" of accomplishment.

Growth of the United States.

What have the American people done in 50 years? Marvellous accomplishments, made possible by the wonderful transportation system in moving food, fuel, raw materials, finished products, and people in vast quantities and over great distances.

TABLE 3.—STATISTICS OF RECORD OF PROGRESS OF THE UNITED STATES, 1870 and 1920.*

Item.	1870.	1920.	Ratio. 1870:1920.
Wealth	$30 068 518 000	† $300 000 000 000	1:10
Wealth per capita	779.83	+ 2 839	1: 3.6
Public debt, less cash in Treasury (July 1st)	2 331 169 956	24 330 889 731	1:10
Public debt, per capita	60.46	228.63	1: 3.8
Interest bearing debt (July 1st)	2 046 455 722	24 061 095 361	1:12
Annual interest charge‡	118 784 960	1 016 592 219	1: 9
Interest per capita‡	3.08	9.55	1: 3.1
Total money in circulation (July 1st)	676 284 427	6 087 555 087	1: 9
Total money in circulation per capita (July 1st)	17.51	57.21	1: 3.3
Total value farm animals	1 518 465 000	8 507 145 000	1: 5.6
Production of Principal Commodities:			
Wool (pounds)‡	162 000 000	308 507 000	1: 1.9
Wheat (bushels)‡	235 884 700	787 128 000	1: 3.3
Corn (bushels)‡	1 094 255 000	3 232 367 000	1: 3
Cotton (500-lb. bales, gross weight)	4 024 527	12 987 000	1: 3.2
Rice (pounds)	54 888 880	§ 1 491 944 444	1:27
Sugar (beet) (pounds)	‖ 806 000	1 452 902 000	1:1803
Sugar (cane) (pounds)	87 043 000	244 250 000	1: 2.8
Gold (dollars)‡	$50 000 000	‖ $49 509 400	1: 1
Silver (dollars—commercial value)‡	$16 434 000	$57 420 325	1: 3.9
Aluminum (pounds)‡	.....	41 375 000	.....
Cement (barrels)‡	.....	96 944 000	.....
Coal (long tons)‡	29 496 054	576 431 250	1:20
Copper (long tons)‡	12 600	539 759	1:43
Lead (short tons)‡	17 830	476 849	1:27
Mineral waters (gallons sold)‡	.....	40 000 000	.....
Natural gas (dollars)	.....	(1919) $162 000 000	.....
Petroleum (gallons)‡	220 951 290	18 622 884 000	1:84
Phosphate rock (long tons marketed)	.....	4 103 982	.....
Iron ore (long tons)‡	3 031 891	69 558 000	1:23
Pig iron (long tons)‡	1 665 179	36 925 987	1:22
Steel (long tons)‡	68 750	(1919) 34 671 232	1:504
Total minerals (dollars)†	$218 598 994	$6 707 000 000	1:31
Manufacturing Industries of United States:			
Cost of material used (1860)	$1 031 605 092	(1914) 14 368 088 831	1:14
Value of products (1860)	1 885 861 676	" 24 246 434 724	1:13
Manufactures of cotton (value) (1860)	115 681 774	" 701 300 933	1: 6.1
Manufactures of wool (value) (1860)	73 454 000	" 464 249 813	1: 6.3
Manufactures of silk (value) (1860)	6 607 771	" 254 011 257	1:38
Total expenses public schools (1870)	63 396 666	(1918) 763 678 089	1:12

* "Statistical Abstract of the United States", 1920.
† Estimated.
‡ Calendar years.
§ Figures include only about 45% of California crops.
‖ Approximate.

The population, production, wealth, and comforts have increased. Schools, colleges, libraries, hospitals, and attention to the sick and poor are all on a much larger and better basis, due to the growth of the country and the use of its resources; and in all this, the railroads were and are absolutely essential, but they have not prospered accordingly.

RAILROAD TRANSPORTATION FURNISHED.

This development has been possible by the movement of an enormous number of tons of freight and passengers, as shown by Table 4.

This development has furnished sufficient passenger transportation, in a reasonably good year, to give every man, woman, and child a trip of more than 350 miles, and enough freight transportation to haul between 3 000 and 4 000 tons 1 mile for each, which is equivalent to handling more than 10 tons of freight 1 mile every day in the year for every man, woman, and child in the United States.

TABLE 4.—Production and Consumption of Transportation.

Fiscal year.	Population.	Tons per mile.	Passengers per mile.	Tons per mile per person.	Passengers per mile per person.
1900.....	76 129 408	141 599 157 270	16 039 007 217	1 860	210.7
1905.....	84 219 378	186 463 109 510	23 800 149 436	2 214	282.6
1910.....	92 267 080	255 016 910 451	32 338 496 329	2 764	350.5
1915.....	99 340 635	277 135 000 000	32 475 000 000	2 790	326.9
1917.....	102 172 845	*398 263 061 787	*40 099 757 819	*3 899	*392.5
1918.....	103 587 955	*408 778 061 079	*43 212 458 079	*3 946	*417.1
1919.....	105 003 065	*367 161 370 571	*46 838 165 980	*3 497	*446.1
1920	106 418 175	*413 673 000 000	*47 966 000 000	*3 887	*450.7
1921†....	107 833 284	*324 000 000 000	*37 700 000 000	*3 005	*349.6

* Calendar year.
† Estimated.

Safety.

Thirty years ago, the average traveler took eight railroad trips per year, whereas now he takes twelve trips. Then, he averaged 24 miles per trip, and now 38 miles. Notwithstanding that he rides more frequently and greater distances than formerly, yet the danger to life is far less than it used to be.

In 1920, the railroads of the United States carried about 1 300 000 000 passengers, with 1 killed for every 5 673 000 carried. In a total of 472 000 000 people carried in 1889, the death rate was 1 in 1 523 000. The danger to life of railroad travelers in 1920, therefore, was less than one-third of what it was in 1889, most of the reduction accruing since 1907. To be sure, there have been years when the proportions varied, but the general trend throughout the whole period has been decidedly toward the increasing safety of the traveling public.

TABLE 5.

Period.	Net railway operating income.	Property investment.	Percentage of rate of return.
Year ended June 30th :			
1911.....	$ 724 184 708	$14 246 167 475	5.08
1912.....	708 484 383	14 632 497 022	4.84
1913.....	787 610 435	15 284 763 489	5.15
1914.....	661 018 147	15 842 127 273	5.17
1915.....	683 104 833	16 257 146 632	4.20
1916.....	984 872 959	16 688 440 056	5.90
Year ended Dec. 31st :			
1916.....	1 040 084 517	16 884 440 038	6.16
1917.....	934 068 770	17 762 152 127	5.26
1918.....	638 568 608	18 213 629 613	3.51
1919.....	454 984 953	18 529 749 653	2.46
1920.....	21 661 782	19 134 000 000	0.11
1921*.....	542 409 582	19 694 622 226	2.75

* Twelve months ended September 30th, 1921. The returns for this period include fifteen large switching and terminal companies.

The Inadequate Pay of the Railroads.

As an abstract question, any fair-minded man will say that "the laborer is worthy of his hire", and that the railroad should be compensated justly and even generously for the service rendered, so as to provide for the future when

more and better transportation will be needed. What has been the result? In Table 5 is shown the rate of return to the railroads for the period 1911-21, which return is wholly inadequate, if private ownership is to be sustained and if Government ownership is to be avoided.

At present, the country and the railroads are in some distress; agriculture is in trouble, industry is slack, and business is poor. The railroads realize this, but those who are now advocating that one way to bring about a resumption of business activity is to reduce the railroad income still further should be careful.

Economic Adjustment, Inflation, and Deflation.

The speaker would like to say a word about "deflation" and "inflation". During the World War, agriculture and industry generally, and wages, were more or less inflated, and much higher prices were received for food, fuel, and articles produced, than prior to the war, and wages were increased. Farmers and farm plants improved their condition very much, and agricultural lands were sold at never before dreamed of prices. Securities representing manufacturing plants were much higher in value and in price. A great many manufacturing enterprises were able to put their prices during the war on such a basis that there were large profits, either in money or in additions to their plants and facilities, so that they are now in a much better position than before the war.

Every one will admit that there was a period of inflation and of considerable prosperity for the farmer, the manufacturer, and the wage earner. There was not, however, for the railroads. They received a scanty living through payments from the Government, in order to keep them going; but, unfortunately, their properties were returned to them deteriorated as to their physical condition, demoralized as to their personnel, and with many new and difficult methods of management and high wage costs imposed.

These conditions, growing out of war and Governmental control, are being corrected slowly by the hardest kind of work between the managements and the men, and with the Governmental agencies, which, under the Transportation Act, have the final power in each of these matters.

All Must Share the Burden of Deflation.

Deflation is taking place in many directions, and all must bear part of the burden. The farmer has felt the full force of this, and from the prosperous condition in which he lived during the war, he has had to come down a long way. This naturally hurts him, and he, not unnaturally, looks around for relief.

The railroad officials believe that agriculture is one of the fundamental industries in the United States, and that they should help it by a moderate reduction in rates as one step in trying to better the existing situation, not because they can afford to do it out of their present returns, but on the ground of helping out.

Manufacturing enterprises are also deflating, and the same may be said about the jobbing and distributing business. It is a slow and difficult process,

but, again, these two classes of human industry had, during the war period, the chance for great prosperity, large profits, and great additions to their plants, and many of them availed themselves of that opportunity, but the railroads did not have that chance.

What is generally described as "labor" has already been hurt by unemployment and by some reduction in wages, which, however, has not yet affected the great transportation interests to the extent it should, if, what some of the people of the country think is needed, namely, lower transportation charges, are to be obtained.

It would seem as if labor, through its great leaders, ought to recognize the fact that this deflation process is going on, admit that it must come, and allow a lower unit of wage in many directions, not only on the railroads, but in the mines and in the building trades, all of which would mean the employment of more men, and a lower unit price on many articles that are used by all, and thus tend to reduce the cost of living and help to break the present endless chain of expense.

Mr. Julius Kruttschnitt, in his interesting and readable article in the *Atlantic Monthly* for January, 1922, makes the following statement about the direct labor costs to the railroads, as the result of the Adamson Law, other Federal and State laws of a restrictive character, the war, and the practices developed during Federal control and action by the Labor Board:

"The labor costs of Class I carriers were 113 per cent. higher in 1920 than in 1917, preceding Federal control; and if the increased wage-scale had been in effect during twelve, instead of only eight, months of 1920, the increase would have been about 125 per cent. The Government during its control allowed gross revenues to increase less than 54 per cent. Labor costs have risen since the Government took charge of them in 1916, under the Adamson law, from $1 468 576 000 to $3 698 216 000, the total amount paid to labor during 1920 being very nearly sixty times the $61 928 626 of net income yielded by the operation of the railroads.

"The history of the direct labor increase is interesting and important.

"The labor bill of Class I carriers in 1916, before the Adamson law took effect, stood at	$1 468 576 394
In 1917, when the Adamson law was in effect, the labor bill was	1 739 482 142
An increase over 1916 of	$270 905 748
This was increased by the Railroad Administration in 1918 to	$2 613 813 351
Or an increase over 1917 of	874 331 209
This was further increased by the Railroad Administration in 1919 to	2 843 128 432
Or an increase over 1918 of	229 315 081
This was further increased by the Railroad Labor Board in 1920 to	3 698 216 351

or $10 132 000 for every day of the year.

"The increase in labor alone, from 1916 to 1920, was $2 230 000 000—nearly equal to $2 357 000 000, the total operating expenses of all Class I roads in 1916, which include, not only cost of labor of every description, but cost of materials, fuel, depreciation, loss and damage to freight, injuries to persons, insurance, and the rest.

"After the return of the railroads to their owners, they were made to perform the greatest transportation task in their history. They moved more freight and passengers, loaded their cars more heavily, and moved larger train-loads. That it cost too much to do this was due, as shown, almost entirely to causes beyond the railroad managers' control."

The Railroads as Buyers of Materials.

A reasonable policy of labor deflation would be extremely helpful, because the wage bill of the railroads is so large a part of their total expenses and they are such heavy buyers of materials, the cost of which is made up so largely of labor. When they are on the up-grade, the railroads are very large buyers—perhaps the largest buyers—of many articles, which means employment to men and a greater demand for food and other articles.

Rates.

The country pays a big bill—through rates—for the transportation service furnished by the railroads, but the country is a big country, and the amount of transportation furnished is very large. The production of it is extremely costly, and, to-day, there is no adequate return to the plant that furnishes the service.

The returns, after expenses and taxes for 1921 (partly estimated) have been paid, will not exceed $616 000 000, not quite enough to pay the fixed charges of about $640 000 000, including the interest on bonds so widely distributed among individuals and held by insurance companies, savings banks, and charitable institutions, thus leaving nothing for improvements or the stockholder. This might be all right if the roads had been allowed in the past, as other industry has been, to earn liberally in other years.

This meager result was obtained only by the cutting of maintenance and repair work to the lowest limit consistent with safety. It has been estimated that in 1921 more than $400 000 000 of maintenance and repair work was postponed, and sooner or later, this work must be done.

It should be remembered that for many years prior to the war the general level of rates was not sufficiently high to protect this National industry of transportation, and there was a danger, to agriculture and to all classes of industry, of the supply of transportation not being adequate for the needs of the country. Therefore, to cure that economic difficulty, an effort was made through the Transportation Act and through the rates permitted by the Commission under that Act.

These rates are higher than they were before the war, but they were then far too low for safety, and we are comparing to-day with an indefensible basis, if the country is to continue private operation and ownership of railroads, subject to public regulation.

Critics of the general level of rates should remember and consider how out of balance rates were before the war and that, to-day, although some may be too high, the general level of rates approved by the Interstate Commerce Commission in 1920, with numerous adjustments made since, is not too high, considering what the railroads have to pay for wages, fuel, and supplies generally; and it should be remembered that the increase in the United States, because of

changed conditions, has not been nearly as great as the increase in transportation charges in many other countries which have suffered from the same world-wide conditions, because of the increase in wages and the price for fuel and materials used by their respective transportation systems.

Rates, in Earlier Years, too Low.

The railroads of this country have always believed in making as low rates as they possibly could, and it has been thought by some that they went too far at times. In fact, the Transportation Act recognized that danger by giving the Commission power to name the minimum as well as the maximum rate. The railroads want as low rates as they can afford, because of the great distances and because they want the widest distribution and the greatest exchange of all kinds of commodities.

The owners and managers, however, of these great railroads are trustees, holding these properties for service to the public, as well as to earn a return, and, in making their rates and in presenting facts to the public about their business, about the charges they make for service, they must realize that they have a paramount duty in maintaining a safe and adequate transportation machine, and in managing it honestly, efficiently, and economically.

To permit wastage through inadequate rates means a deterioration of the properties within their charge, which will tend to prevent their serving the public adequately; and to permit such wastage is not honest, efficient, and economical, any more than it is to permit wastage in labor, if they can control it, or in the use of material, or in any other form of management.

Many Other Factors to Consider Besides Rates.

Rates, of course, have a bearing on the development and success of agriculture and industry; but they are not the only important elements in the problem, that must be considered before improved business conditions obtain.

For example, to-day, the labor cost of building a house is far more important than the rate on the material itself, and the labor cost is a much greater deterrent to solving the important housing problem of the country than the railroad rate. This labor cost goes back into the lumber, the steel, the iron, the brick, the cement, into the railroad rate, and, it is evident, in the wages received by the men actually engaged in placing the material, after it has been put on the ground, in the form of a house.

It is very important, not only for the railroads, but for the public, to be just and without prejudice, and to realize that there are a great many factors besides rates which are affecting business.

A sweeping reduction in all rates would not, in the speaker's judgment, at this time increase business, for many other things must be settled before we obtain what all desire, a complete revival of industry in this country.

The speaker refers to such matters as the various foreign questions which seem to be in a fair way of adjustment; the tariff question, which is most difficult; the tax question, which has been partly settled and which is even more difficult than this railroad rate question, and the general financial condition of the world.

The Transportation Act.

During some of the time since the passage of the Transportation Act of 1920, business has been depressed and disturbed, and it can truthfully be said that the Act has not had a fair trial. In 1920, the cry of the country was not about rates, but about the inability of the railroads to handle all business promptly. That condition is likely to exist again when business resumes, if proper steps are not taken.

The country, in that Act, declared for a National railroad policy, and the supreme power of the nation, through the Commerce Commission and the Labor Board, over questions of dispute as to rates and regulations thereof and wages and working conditions, respectively; also for a policy of self-support for the railroads out of the rates charged for transportation furnished to the public. The public, after this long debate, turned away from the doctrine of supporting their transportation system through taxation, and also turned away from Government ownership. It will be most unfortunate for the country to turn back to Government operation or ownership, because of the present difficult conditions from which it will emerge in due time.

Economies in Transportation.

Improvements in transportation practice have been continuous for a long term of years, and the total accomplished is likely to be lost sight of. Not so many years ago, all the land transportation in the country was furnished by packing on the backs of men or animals; to-day, there are being handled with the aid of less than 2 000 000 men, more than 400 000 000 000 ton-miles of freight each year. The physical effort to do this represents, under the old methods, the labor of about 1 250 000 000 men, and the cost to the public has decreased from $3 to about 1¼ cents per ton per mile, based on 1921 returns.

Of course, it would be impossible to apply this amount of man power, but the comparison is given to show how the efficiency of the railroads has released untold man power for the great development of the country in many other directions. It may be argued that highway transportation would have developed even more than it has if the railroads had not been here and the country, therefore, served by highways and waterways. Without the railroads—the primary form of transportation in the country—the motor vehicle, the gasoline to propel it, and the hard surfaced roads never could have been developed, except to a limited extent.

Every progressive railroad manager knows that some further economies are possible, but they depend largely on two factors that cannot be made effective at once: First, the spending of money for capital account in the creation of better facilities; and, second, the actual price paid to the individual workman and the development in him of a spirit that it is his duty, as an American citizen, to do all he can to bring down the cost of transportation, because of the help such reduction will be in working out the soundest possible general economic scheme for the whole country. This same idea ought to be applied to those who work in mines, factories, and forests, because fuel and material prices affect all and very largely depend on the efficiency of the individual man and the wage paid.

The engineer is peculiarly in a position with his trained and accurate mind to help in determining the wisest form of improvements in transportation facilities and practices to be made (if and when money can be obtained) for increasing economies of operation. Among economies that are not completely developed are:

1.—Further decrease in the physical resistance to be overcome so as to lessen the tractive power necessary to haul a ton of freight.

2.—Improved location and design of terminals both for handling carloads and merchandise; improved water and fuel stations.

3.—Great economies in the production and use of power from steam and electricity, whether obtained from fuel or water power.

4.—Improved design of locomotives and cars so as to have the maximum of capacity and strength with a minimum of dead weight.

5.—A better supply of well-designed repair plants equipped with high-powered and rapid-working tools in all shops and roundhouses, where mechanical work is performed.

6.—Improved methods of receiving, storing, and distributing the great quantities of materials and supplies that are used each day by the railroads.

7.—The elimination of waste which is prevalent in all walks of American life.

The Future.

It is not beyond belief that within 50 years there will be 250 000 000 people in the United States. They can be well cared for if every one will co-operate and work. Vital to their well-being, however, will be an adequate transportation machine. To-day, the country has one made up of the railroads, seaways, waterways, highways, and motor transport, representing great values, possibly $60 000 000 000, or next to agriculture as an element of National wealth. Of this, the present railroad system, on any fair basis of allowance for good will, strategic position, experienced organization, and all the elements that enter into values of other classes of property, is probably worth $25 000 000 000. When there are 250 000 000 people, a ton-mileage of even 3 000 per annum instead of the present 4 000, to give them as good a living as is being furnished to-day, will mean the movement of 750 000 000 000 ton-miles per year plus the added passenger travel—double that now provided.

Thus, the present great transportation machine must be enormously added to, possibly doubled, not in mileage, but in capacity, and co-ordinated with other forms of transportation. Great sums of new capital must be obtained, probably more than $25 000 000 000, and this can only be obtained if there is safety for the principal and a fair annual return. The engineer is particularly interested in having this work go on, because of the important part he will play in it, as well as in the development of economies in the existing system.

The country and the world are somewhat in the "dumps", but times will change and conditions will be better. The speaker believes that the United States is on the threshold of the most glorious period in its history, and that Americans can take a place in the history of nations never before achieved if they will only think right, work hard, and use carefully their great powers and resources; and the transportation machine should be kept at all times adequate to the growing needs of the country.

The Engineer and Social Progress.

The railroad system of the country has been and is a great engine for advancing civilization. The engineer has played a most important part in creating and operating this engine. With the great development in the United States, however, and the crowding of people into cities, have come perplexing social problems which must be attacked and solved if the country is to achieve its highest destiny. Some of these are:

(*a*).—*Better Use and Conservation of Natural Forces and Resources.*—Nature has been generous to the United States and the people have not been obliged to save in the past as much as they must in the future. The engineer, by perfecting the processes for developing and conserving resources, can help to bring about better living conditions and promote National health and National safety. Think of the possible development in the next fifty years along the lines of fuel conservation, use of cheaper forms of coal for power, water-power development, control of the flood waters in valleys, and control of water in the mountains for irrigation. The better use of natural forces and the elimination of waste in use of natural resources are most important.

(*b*).—*Decentralization of Population.*—In 1880, 14 358 167 people lived in cities and towns and 35 797 616 in the country. In 1920, this had changed to 54 304 603 for cities and towns and 51 406 017 in the country.

Cheaper fuels and wider distribution of power will permit a wider use of mechanical devices on the farm, in the small home, and in the small factory, and by increasing comfort and convenience in the country, check the drift to the city.

(*c*).—*Better Rewards for the Farmer.*—The cost of living can be reduced if there is a National policy leading to the better use of resources, and more people willing to produce food.

Agricultural and farm life however, must offer attractions and rewards to the young people equal to what they think can be obtained in the cities.

(*d*).—*The Relation of Labor and Capital.*—The distribution of the annual increment of wealth in the country very properly is a subject that engages the attention of every one. Each one naturally desires to obtain all he can consistent with regard to others. Out of this natural effort great organizations of labor and of capital have developed. Such organizations are all right if wisely and honestly directed. Labor unions are all right when they are conducted so that waste is eliminated, and they are an efficient and economical instrument of society as a whole. When they cease to be that, their usefulness is gone and society will get rid of them. There is a place in this country for wisely directed labor unions, but no place for labor tyranny.

Society some time ago discarded the club, the sword, and the pistol as weapons for the settlement of personal disputes. Society is getting ready to discard the strike as a weapon for settling industrial disputes, because it is inhuman and uneconomical, the sooner the better for the good of all.

The engineer is of great importance because of his technical knowledge and trained mind. He is of almost greater importance, however, as a citizen, because he has been trained to be accurate in thought, his calculations must

check out, or he fails. He can be of great value in working out some of these social problems, and he can and should counteract the loose thinking and talking that is more or less prevalent, resulting in the advocacy in legislative halls, on the stump, and in parlors, of foolish doctrines that never have worked and never will.

The majority of people in the United States, as individuals, are sensible, hard-working, and home-loving; but, collectively, they are sometimes led astray through lack of accurate information and clear thinking. The World War developed a spirit of extravagance and unrest, and there is a noisy minority doing a great deal of talking and advocating what, for want of better names, are known as Radicalism, Socialism, Bolshevism, and the use of remedies that may do more harm than good. The engineer, as a citizen, can use his trained mind to help prevent the theorists and adventurers from leading the country astray so that we may jump out of the "frying-pan" of admittedly difficult conditions into the "fire" of much worse ones.

The engineer, accustomed to basing conclusions on accurate facts, can help create a public opinion: That an adequate transportation machine is a necessity; that it cannot be had without paying for it; that the present railroad system was developed by giving the initiative of the American man some chance to work and to obtain rewards; and that better results can be obtained in the future if that initiative is allowed once more to be used without all the hampering and restrictive laws of Nation and States that have been put on the statute books in the last twenty-five years.

Railroads are common carriers of people and property; they are not common carriers of all the economic troubles of the country. These cannot be cured by reducing rates, by ruining the railroads, and perhaps forcing Government ownership to which this country is opposed. Give brains, courage, and management a chance once more. Declare a ten-year holiday in the ceaseless investigation of the transportation question, and let the undivided attention and energy of owners, managers, and employees be devoted to maintaining, operating, and perfecting this engine of civilization—the wonderful railroad system of the United States.

RAILROADS AND THEIR EMPLOYEES

By W. N. Doak,* Esq.

Some of the world's greatest men have been engineers and engineers have blazed the way for civilization and progress through the different stages of the world's development. The achievements and progress of mankind are so interwoven with the work of civil engineers that it may properly be said they have been the arbiters between nations and subdivisions thereof.

The speaker will advance only a few observations on the subject of the relations between the railroads and their transportation employees. These conclusions are the result of more than twenty years' observation and several years' practical experience, most of which time the speaker has had the pleasure of personal contact with both railway managements and railway operatives. What is said will be devoid of any tinge of partisanship and will be a statement of a personal conviction.

The relation between employer and employee engaged in the transportation industry should be the most cordial, because in reality they are both employees of this great institution engaged in a public service. Both should be treated fairly by the public, and impartiality should guide in the consideration of all questions affecting either.

Disputes between railway managements and the transportation employees are largely unnecessary and almost wholly preventable through a closer relation and a more friendly feeling, combined with the exercise of calm judgment and the use of common-sense principles. The greatest menace to industrial peace is the lack of proper understanding between employer and employee, which, in many instances, is the result of thoughtlessness and carelessness and aggravated by undue outside interference and improper design.

Differences often spring from the poisoned well of ill-advised propaganda, which is either devoid of all truth or is inspired by the greater menace, half-truths. Half-truths are far worse than whole falsehoods. The speaker is convinced, therefore, that what is needed above everything is the exact truth in all instances. Truth accurately arrived at by and through the process of calm, unbiased investigation by practical, experienced investigators, and the facts thus obtained applied in the most practical manner.

Neither unscientific conclusions nor half-hearted investigations by impractical unreliable investigators can result in other than damage and harm. Harm to the one must eventually result in harm to both management and men and in the end be dangerous to the industry. Temporary advantages to either will ultimately result in disadvantages to all parties at interest. Both employer and employee, and, in fact, the industry itself, have been the victims of unjustifiable assault in many instances by the public through the lack of proper understanding which, in turn, was the result of partisanship and improper publicity. The money spent in publicity and the energy expended to gain a temporary advantage should have been used to arrive at truths and in the dissemination of facts to the public instead of the ill-advised course pursued

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in so many instances. An unbiased judgment compels the speaker to state that his observation is that the industry is all right, the management is all right, and the employees are all right, if permitted to follow a normal and proper course, and by the united efforts of all the greatest protection is guaranteed to the transportation industry.

Probably one of the pastimes most largely indulged in by the average citizen is the discussion of railroad problems. Bitter partisanships have been made, and, in most instances these partisans cannot give any logical reason for being aligned with either the railroads, the managements, or the men. The reason for this is largely due to ill-advised publicity in the form of paid advertisements, editorial columns, or utterances from the public platform by intense partisans. In most instances, those engaged in the dissemination of this propaganda are wholly uninformed as to the exact facts, or deliberately refuse or neglect to state all the facts. Suppose, for the sake of comparison, a dispute should arise as to the boundary between two States, which resulted in an intense partisanship, and all kinds of propaganda were used by the parties at interest. No attention whatsoever would be paid to any such propaganda unless and until experienced engineers had run the line and passed judgment on this question, thereby arriving at the facts. This has not been the manner in which railroad questions have been judged, nor has it been proposed that they be dealt with in any correct manner. The point is that any other dispute besides a railroad labor dispute and certain other railroad questions, is always settled by experienced and practical men, whereas it is proposed for the public to settle these disputes with impractical and inexperienced men.

After a half century of agitation, experiment, and countless discussion, railroad men find themselves drifting in a more disturbed and uncharted course than at any time in twenty years, when it comes to the adjustment of railroad labor disputes. Strikes and lock-outs, mediation, conciliation, arbitration, Courts, boards, and other plans, have been tried, but the present time marks a very unsettled condition.

Railroad men have the experience of the United States and also the benefit of the experience of almost all civilized countries. This affords an opportunity for comparison and after so doing, voluntary methods have been found to be the most effective and far outweigh those of other countries. The most effective remedy has been voluntary mediation, conciliation, and arbitration. The Erdman Act and its amended Act, known as the Newlands Law, proved by far the greatest means of accommodation of any other plan tried in the United States or any other country. This Act was suspended temporarily during the period of Federal control, and all questions were settled by boards of wages and boards for the adjustment of other disputes, bi-partisan in character, with the Director General of Railroads as the referee in case of deadlock. With the termination of Federal control and the enactment of the Transportation Act of 1920, the United States Railroad Labor Board was created, and provisions were made for voluntary bi-partisan adjustment boards, the Labor Board to have jurisdiction over all disputes that were not adjusted by other means and to act as referee in case of deadlock by adjustment boards. This method is still in existence and the speaker does not believe it has been sufficiently tried out

to form a correct conclusion as to its effectiveness. He has believed, however, that no tri-partite board would ultimately be successful because of the inexperience and lack of knowledge on the part of one-third of the membership. Furthermore, the element of partisanship is not eliminated when there is any possibility of having a third party to whom the responsibility may be passed.

Some countries have tried Industrial Courts, in fact, one of the States is now trying out this plan. These did not work satisfactorily in any country where they were tried, and they have not proved adequate or desirable as a method of accommodation for disputes involving interstate commerce employees in the United States. The Kansas Industrial Court did not prevent the outlaw strike of railroad employees in 1920, nor did it prevent the strike of miners in that State. Of course, a few men were sent to jail, but this did not keep the miners on the job, nor can any Act be passed in this country, where free labor is guaranteed by the Constitution, that will keep individuals on the job. The Industrial Courts of Australia and the Australasian countries eventually proved a failure.

The Industrial Disputes Acts of Great Britain and of Canada have not proved as effective in the adjustment of disputes as the laws in the United States. As a matter of fact, in the same dispute involving men on both the American and Canadian sides, an accommodation was had under the Newlands Act in the United States and a failure under the Canadian Industrial Disputes Act. In fact, peace was secured under the American laws and the Canadians had a strike and industrial war.

Compulsory arbitration laws have been enacted in a few countries and they have failed for the reason that individual freedom prevents men from being compelled to work against their will. No such law could be effectively enforced in the United States, because the greatest freedom is accorded the individual working man in ceasing his employment when he pleases.

Another experience in this country in the adjustment of disputes between the railroads and their employees that has proved to a degree very satisfactory is through the agency of voluntary bi-partisan boards of adjustment, on which employer and employee were equally represented. Also, a plan of this kind was tried for the adjustment of disputes between the anthracite coal operators and the miners, following the anthracite strike during the Roosevelt Administration, and this plan has been in effect since that time preserving industrial peace in this industry. The speaker believes the printers and their employers adopted a somewhat similar plan which has been in effect for a number of years, but in each instance, of course, a referee was selected in event of failure of agreement or in case of deadlock.

This brings up a discussion of the merits of various plans and to conclusions as to how disputes should be settled between the railroads and transportation employees.

Conciliation.—Conciliation has been valuable in getting the parties to meet and settle their differences in conference after one or both had refused conference or declined to proceed further with negotiations. This has been very helpful and has paved the way for many settlements, either by direct

negotiations, through mediation or arbitration. It should be encouraged and stimulated, if for no other reason than that it results in direct conferences between the parties and acts as a stimulant to the principle of settling differences around the conference table at home.

Mediation.—Mediation has been the most potent factor of the many principles applied to the railroad industry. In many instances after conciliation had failed, mediation was successful in bringing about an adjustment of the dispute by the mediators meeting the parties separately or collectively and aiding them in composing their differences. In other instances, arbitration has resulted from the efforts of mediation and proved to be successful. High-class mediators should be provided by law or by voluntary arrangement, preferably by law.

Arbitration.—Voluntary arbitration has been of mutual benefit in the adjustment of controversies on the transportation line. However, in each instance, these have been voluntary and they should be so continued and only resorted to after every attempt at conciliation and mediation has failed. Both railway managements and employees are committed to the principles of arbitration. In the speaker's opinion, it should be the least resorted to of any other plan, and when taken advantage of, the rarest care should be exercised in the selection of members of the boards, especially in the selection of neutral members of such boards. Arbitration has largely failed or fallen into bad repute because of unwise selection of neutrals and in the administration of arbitration laws rather than from any lack of merit in the principle. Some method for the creation of voluntary arbitration boards, to be used when desired, should be provided.

Bi-partisan Boards of Adjustment.—Bi-partisan boards of adjustment for transportation employees originated after the passage of the Adamson Eight-Hour Law. It was necessary for some one to apply the principle of the 8-hour day to the several hundred wage agreements between the railroads and the four transportation organizations, and as a result a committee of eight was agreed on, consisting of a railway representative from each of the different territories and a representative from each of the four transportation organizations. This Commission handled approximately 30 000 cases during the year it was in existence, and, fortunately, agreed on each case. With the commencement of Federal control, the question arose as to the handling of not only 8-hour questions, but all other disputes, including wages, discipline, etc. It was then arranged to appoint a board of eight members to handle all grievance matters other than wage disputes for the transportation men, the Director General of Railroads to be the referee in case of deadlock. Afterward, similar boards were created to handle such matters for other classes of employees. After the Lane Commission handed down its wage award, it was agreed to appoint a committee to handle wage questions, with the right to recommend changes in wage rates to the Director General of Railroads.

The adjustment board for the transportation employees handled more than 3 000 disputes without a deadlock. The other adjustment boards likewise did good work. This experience brought the industry up to the close of Federal control in good shape, and with the benefit of a most valuable experience.

The Transportation Act of 1920 then provided for voluntary adjustmen boards for the adjustment of disputes other than wage questions and a labor board for the adjustment of wage disputes and to act as referee in cases of deadlock by adjustment boards. Boards have been recently created for some of the railroads and the transportation employees and are now functioning. These should be continued and encouraged.

Conclusions.—What is desired above everything is industrial peace coupled with fairness and proper treatment alike to the railroads, the management, and the employees. The speaker, therefore, would recommend the following:

1.—Every effort should be exerted through direct conference and meetings of the employees with their employers in an effort to adjust any controversy between the two.

2.—Agencies to provide mediation and conciliation with the right to propose voluntary arbitration should be re-established at the earliest possible time, and rare judgment should be exercised in the selection of high-class men for such positions.

3.—Regional adjustment boards, bi-partisan in character, should be established and all railroads and their transportation employees represented thereon, to which would be submitted all disputes that could not be adjusted otherwise.

4.—The United States Railroad Labor Board should be continued until a fair trial has been accorded it and an accurate estimate made as to its value, this Board to adjust such wage disputes as are not adjusted otherwise and to act as a referee in case of deadlock by adjustment boards.

5.—The practice of agitating, either through the public press or otherwise, questions that are likely to disturb the public mind and cause alarm among the people regarding railroad questions and railroad labor matters should be discouraged and if possible entirely eliminated, and in lieu thereof agencies established for a fair impartial investigation of all such questions by which to arrive at accurate facts.

6.—The public, the railroads, the management, and the employees should unite in a campaign of education that would restore confidence in the industry, through the dissemination of nothing but facts in dealing with questions affecting the railroads.

With these few suggestions, the speaker hopes the future holds in store an unbroken era of industrial peace and likewise a revival of confidence and prosperity in the railroad industry. With prosperity in this important industry all interests must prosper, and the railroads are usually one of the most accurate business barometers.

RAILROAD TRANSPORTATION AND OWNERS OF RAILROAD SECURITIES

By F. A. Molitor,* M. Am. Soc. C. E.

Of recent years a new element has entered the railroad problem: the security owners. The narrowing margin in the net operating income of the railroads, from which interest on bonds, dividends on stock, and improvements to property must come, occasioned such misgivings that the National Association of Owners of Railroad Securities was formed for the purpose of protecting their interests.

The date, September 6th, 1916, when the Adamson Act was unfortunately passed to avert a strike threatened by a strong combination of railroad unions, was an important milestone in the history of American railroads. It marked the beginning of the decline of the credit of the railroads of this country and evidenced the great political strength of the railroad unions, working for the first time in a strong combination.

From that date, the investing public ceased their former interest in, and purchase of, railroad securities. This withdrawal of interest and support on the part of the public from the purchase of railroad securities, is evidenced by comparison of sales before and after 1916. From 1910 to 1915 the percentage of sales of railroad bonds to the total sale of all bonds on the New York Stock Exchange averaged 93 per cent. In 1916, the sales of railroad bonds declined to 73%, and, in 1917, to 44½ per cent. During 1918 and 1919, the sales declined to 11% and 12%, respectively, which low percentage, however, was caused by the large volume of Liberty Bonds sold during these years. In 1920, the percentage of sale of railroad bonds to all bonds was 20½, and, in 1921, it was 31.

Some well-informed investors commenced even in 1916 to dispose of railroad securities, but institutions such as savings banks, insurance companies, etc., that held large amounts of railroad bonds, were not able to dispose of their large holdings without a further and more serious disturbance of values, so that to-day it is probable that the railroad bonds are largely held by these institutions and trust estates, and, relatively, a few individual investors.

The very direct interest in these bonds by saving bank depositors and insurance policy holders is manifest. The savings banks and other banks of the United States hold about $1 700 000 000 par value of railroad bonds and there are 10 000 000 depositors in these banks. The insurance companies hold about an equal number of railroad bonds ($1 520 000 000). The policy holders number 34 000 000. In all, this represents about 32% of the total railroad bonds outstanding. These 44 000 000 people can wield a great political power and, in the last analysis, control the political situation in the railroad problem, if they can be aroused to act together through their organization.

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The withdrawal of the support of the investing public from the railroad security investing field between 1916 and the entry of the United States into the World War would have produced an immediate financial crisis, if it had not been for the enormous business that came to the railroads by reason of the purchase in this country of munitions of war by the Allies. Indeed the years 1915, 1916, and 1917, commonly called the "test period", were years of considerable prosperity to the railroads, due principally to stimulation of business caused by war demands. During these three years, the average net return on the investment account of Class I railroads was 4.52%, while the net return for the fiscal year 1921, with December estimated, was only 3.28 per cent.

The taking over of the railroads by the Government, as a war measure, on December 31st, 1917, the speaker has always personally felt was quite unnecessary. They could have functioned as well under some temporary unified control which readily could have been accomplished through the aid of the Government. This would have avoided that Government operation which has done so much to destroy operating efficiency and the consequent credit of the railroads.

Railroad labor, by reason of its efficient organization, had placed on the statute books, beside the Adamson Law, the Full-Crew Laws in various States and other unnecessary legislation which increased operating expenses without any compensating advantages to the railroads or to the public. The railway executives then (in 1917) formed an organization in order to present to the public and to Congress the point of view of the management and stockholders, as well as to handle negotiations with their labor on a common basis, made necessary by the common organization of railroad labor. In September, 1917, it became necessary for the security owners, largely represented in bond holdings, to form an organization so that their own voice and viewpoint might be heard, and their particular interests as owners of the primary securities presented to Congress and the public. Their Association was formed through the personal energies of Mr. S. Davies Warfield. This organization is in effect a bondholders' protective committee for all the railroad bondholders, acting for all securities, and not on behalf of any group or single system. The necessity of their action is due to the decreasing margin applicable to fixed charges and dividends of all the carriers.

The interests of the Security Owners acting for the bondholders, the railroad executives who operate the properties and represent the stockholders, and the general public, are necessarily one.

The Security Owners Association was mainly, if not wholly, responsible for the inclusion in the Transportation Act of 1920 of that provision for a measure of rate-making, which, unfortunately, has been called the "guaranty" provision. It is no guaranty at all. Otherwise, the existing deficit below the 6% return would have to be made up from taxation. It provides that the measure of rate-making shall be such as in the aggregate by groups will yield a 5½ to 6% return on the valuation of the railroads. This provision expires on March 1st, 1922, and many efforts have been made to remove

it from the Act. These untimely efforts have occasioned much concern to the investors, because the provision is a guide or "yardstick" for the making of compensatory rates, heretofore defined under the anomalous words "fair return". These efforts of the agricultural and other interests to obtain the removal of this provision is increasing the alarm of the investor concerning railroad securities. The apparent policy of the Interstate Commerce Commission and some of the railroad managers to yield to this pressure and to decrease rates in the face of the existing low net return adds to this alarm.

After the passage of the Transportation Act many influential public interests of the country, and many of the railroad managers were hopeful, if not enthusiastic, that the Transportation Act would stabilize the policy of regulation so as to enable the railroads to rehabilitate their properties and restore the operating efficiency lost during Federal control. The business depression which affected the railroads during 1920 and 1921 was, of course, unavoidable. It came from world-wide economic causes and to-day much liquidation, has taken place. The railroads have had their full share of this liquidation, as have the large industries, but, unfortunately, while liquidation in wages in the broad industrial field has taken place, it has not, except to a very small extent, occurred in the ranks of railroad labor. They are still receiving within 10% of their peak wages. The lack of liquidation in the ranks of railroad labor is shown by Tables 6, 7, and 8. From Tables 6 and 7 a comparison of indices of average compensation of railroad employees with cost of living may be made. Table 8 gives a comparison of railway wages with some other wages.

TABLE 6.—Average Annual Compensation per Employee in Percentage on Basis of Fiscal Year 1913: Class I Railroads.*

Month.	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
January	....	....	...	....	....	....	....	209	235
February	....	....	....	....	....	....	....	209	235
March	....	....	....	....	....	....	....	209	235
April	....	....	....	....	....	....	....	210	234
May	....	....	....	....	....	....	....	250	234
June	100	107	108	112	....	....	....	249	234
July	....	....	....	....	....	....	....	256	207
August	....	....	....	....	....	....	....	256	214
September	....	....	....	....	....	....	....	256	205
October	....	....	....	....	....	....	....	250	....
November	....	....	....	....	....	....	....	250	....
December	....	....	....	116	132	186	195	250	....

* Compiled from Statistics of Interstate Commerce Commission.

The railroad security owners have also taken more than their full share of this liquidation, as evidenced by the comparison of net return of the past 12 months with the return during the "test period". The Federal Government and the various political sub-divisions have not assumed their share of the liquidation, in that there is no reduction in the taxation of the railroads. Indeed, this item has been increased alarmingly. In 1912, the total taxes derived from railroad property by various governmental agencies, that is,

Federal, State, and municipal, was $109 500 000. In 1921, this has increased to $490 000 000 or 348 per cent. This, however, includes the Federal tax on transportation amounting to about $200 000 000. The abolition of the Federal tax on transportation in 1922 will reduce the amount of taxation by $200 000 000 to $300 000 000 annually, dependent on the volume of business. This item, however, represents the amount paid by the public to the railroads, which is passed on as a payment to the Federal Government. An interesting comparison in this connection is that Class I railroads paid (in addition to the transportation tax) in 1921 to the Federal, State, and other governments about $290 000 000, while the amount they paid in dividends during 1920 to stockholders was $271 000 000, so that the owners of the railroad properties received less than the Government.

TABLE 7.—Cost of Living, in Percentage, on Basis of July, 1914.*

Month.	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
January	....	....	....	....	....	....	....	190	181
February	....	....	....	....	....	....	....	194	176
March	....	....	....	....	....	....	161	195	169
April	....	....	....	....	....	....	....	197	168
May	....	....	....	....	....	....	....	202	166
June	....	....	....	....	....	152	....	203	162
July	100	100	101	109	131	....	172	205	163
August	....	....	....	....	....	....	....	203	162
September	....	....	....	....	....	....	....	199	165
October	....	....	....	....	....	....	....	197	164
November	....	....	....	....	....	165	182	193	....
December	....	....	....	....	....	....	....	190	....

* Compiled from Statistics of National Industrial Conference Board.

TABLE 8.—The Relation of Railroad Wages to Some Other Wages.*

Year.	Railroad union wages.	Seventy-two other union wages.	Farm wages.	Combined crop and livestock prices.
1913	100	100	100	100
1914	105	102	99	103
1915	109	103	100	98
1916	113	107	109	119
1917	129	114	137	185
1918	189	133	171	207
1919	227	155	202	213
1920	272	199	235	185
1921	288	....	192	122

* *Industrial News Survey,* January 7th, 1922, p. 6.

The result of all these happenings is that the credit of the railroads is to-day at the lowest ebb. Unless the railroad problem is removed from politics, and an economic and liberal policy is pursued in regard to the railroads, the result can only be financial disaster or early Government ownership.

In regard to Government ownership, the campaign made in its favor by the railroad unions in support of the so-called Plumb Plan, may be recalled. This did not receive public support. However, since the passage of the Transportation Act in 1920, the general public has been lulled into security

and peace in respect to Government ownership. The undercurrent of political affairs is such that many believe public ownership of railroads is closer to-day than it was three years ago. The effective force that worked in its behalf then, is working as effectively now, and the meager return to the carriers, since the passage of the Transportation Act, is helping the movement along.

The happenings which the speaker has briefly reviewed and which have made for the impairment of railroad credit, may be summarized as follows:

1.—The passage of the Full Crew and other unnecessary laws by several States.

2.—The refusal of the Interstate Commerce Commission to increase rates in 1915.

3.—The passage of the Adamson Law.

4.—The inability of the railroads since 1916 to finance needed improvements, especially terminals.

5.—Government operation during the war.

6.—Increase of wages granted by the Railroad Labor Board in July, 1920, retroactive to May, 1920.

7.—The agitation to abolish the provision of the 6% return of the Transportation Act of 1920.

8.—The decrease in many freight rates below those established by the Interstate Commerce Commission in August, 1920, in accordance with the Transportation Act.

9.—The slowness in reducing wages, which thus far has only amounted to approximately 10 per cent.

10.—The extreme deliberation on the part of the Administration in settling the railroad claims for rentals, deferred upkeep, etc.

The failure to adjust wages to the decreased cost of labor, and to maintain the freight rates to the tariffs adopted by the Interstate Commerce Commission, in August, 1920, particularly, is shown in the operating results of the carriers for the calendar year 1921. With December estimated, the net operating income of Class I carriers was about $610 000 000, or a return of 3.28% on their tentative valuation. They failed by about $505 000 000 to earn the 6% return contemplated by the Transportation Act, under the freight tariffs made thereunder.

Furthermore, this small return was at the expense of maintenance of way and equipment, and is inflated to the extent that the railroads failed to expend a normal maintenance amount. The speaker estimates that about $400 000 000 (or 20%) more should have been spent in 1921 to have properly maintained the railroad plant. Had this been done, the return would have been 2.15% less, leaving a net return slightly over 1 per cent. Therefore, this is the figure the readers are asked to carry in their minds.

These results are no reflection on the excellent management of the individual railroads. Maintenance expenses were necessarily cut to the minimum in the attempt to provide available funds to meet interest requirements. The excellence of the management by the railroad executives of their properties is amply shown by the fact that they have thus far avoided receivership.

It is the duty of the engineer to the public to suggest remedies for economic disturbances. He has been taught by contact with Nature to be constructive not destructive. So the speaker boldly suggests these remedies for the railroad problem:

First.—

A.—Discontinue any further general reduction in freight rates.

B.—Adjust railway labor wages to conform to the cost of living.

C.—Transfer the Railroad Labor Board to the Interstate Commerce Commission.

D.—Continue the 6% return as a measure of rate-making after March 1st, 1922.

Second.—

A.—Employees of any transportation or public utility should be prohibited by statute from striking.

B.—The incorporation of all labor unions, the members of which are engaged in the transportation of any vital necessity to the public, should be insisted on and should be required to file full financial reports with the Secretary of Labor. It should be further required that all ballots be secret ballots and counted under the supervision of a representative of the Department of Labor.

Other measures are necessary to the rehabilitation of the railroads, but on account of the complications and magnitude of some of them, steps leading to their accomplishment must be taken with more deliberation than any of those just stated. Their adoption, the speaker is certain, will appeal to the professional mind, which also will recognize their economical possibilities. These economies, when proved effective, can then be passed on to the public in the shape of reduced rates, and in the speaker's judgment, until these economies are effected, the rates cannot be safely reduced with the present business, and with the present outlook for business in the immediate future.

Of these measures the speaker refers, first, to the consolidation of railroads, in respect to which he has many personal doubts. However, political conditions have become such, and the public demand for consolidation is so strong, that it appears likely that the railroads of the country must be consolidated into a comparatively few systems, somewhat as suggested by the Chairman of the Senate Committee on Interstate Commerce, Senator Cummins of Iowa. If consolidations are based on a sound financial and operating structure, they will automatically cause the unification of the larger terminals.

The second measure is the terminals. The co-ordinating or unification of the larger terminals and their operation under a single management will undoubtedly reduce the capital used therein, as well as save duplication of operation. Store door collection and delivery in some cities, like New York and Chicago, will make for savings in trucking costs.

Third and last is the standardization of the freight equipment of all the railroads and a centrally controlled car-pool.

The speaker is strongly of the conviction that these three suggestions, if they can be carried out, will effect enormous savings in the general operating

account, which, when passed on to the public in the shape of freight rates, will more than satisfy even the most critical shipper.

In closing, the speaker is sure that he expresses the sentiments of the President, and Board of Directors of the National Association of Owners of Railroad Securities, when he states that their endeavor is to save the credit of the railroads, to avoid Government ownership, and to assist wherever it may lie in their power, those able servants of the public, the railroad executives, and this opportunity is taken to state that it is hoped by its Committee to suggest the inclusion of the financial means of their accomplishment.

The Security Owners Association has asked Congress to incorporate the National Railway Service Corporation, which if thus chartered will be enabled, with the assistance of the large security owners of the railroads comprised in its membership, to finance the unification of terminals, the furnishing of freight equipment, and to suggest a method of car-pooling, all of which may be done without profit and with a great saving to the railroads.

DISCUSSION

GEORGE W. SIMMONS,* ESQ. (by letter).—The transportation situation is one of the fundamental considerations of business at the present time. Although the writer is convinced that it can be technically proven that most rates to-day are not the primary cause of low prices or depressions, he is convinced, nevertheless, that there is an insistent public demand that rates shall be reduced. If this demand is not met by a reduction based on intelligent and economic lines, then there is a danger that the public will insist on a reduction anyway. No reduction should be made unless it is by reason of reduced operating costs. The railroads to-day are being economically operated, as compared with other industries. There is still extravagance and waste, largely by having several men to do the work one man could do, and by paying for work not actually performed—all the result of regulations governing working conditions inherited from the Federal Administration. The interests of the people demand that railroads be operated as nearly as possible on a business basis of efficiency and economy, and that the resulting savings be translated to the public in reduced rates.

The sentiment of the public is overwhelmingly against Government ownership of railroads. The writer has recently taken a poll on this subject from about 500 salesmen, whose reports in the past on such matters have been proven practically 100% accurate. The reports show that the only sentiment favoring Government ownership is among railroad employees and organized labor, and among those who believe in Socialism. A few scattering reports indicate that some farmers are almost willing to try Government ownership as a means of lowering rates on farm products. In almost all sections, however, there remains a recollection of Government operation and almost universally the sentiment is "never again!"

Farmers generally are getting so much less for their products than what they are entitled to, in proportion to what they have to pay for other things, that they are inclined to listen to the demagogue who tries to fasten the blame on high freight rates. Farmers generally recognize, however, the necessity for the Federal regulation of car supply and adequate service to carry their products to market, when there is a market. Many of them, however, do not recognize that the chief trouble with the price of American farm products lies in Europe and other foreign countries, and that the freight rate has little if any influence on the quantity of cotton, wheat, and hogs that are exported. The lack of market rather than the transportation charge is the fundamental reason for the low price of farm products.

If railroad rates can be reduced soon, with a corresponding reduction in operating costs, it will do more to start the wheel of prosperity throughout the country than any other one thing. The repeal of the Adamson Law and the readjustment of railroad labor costs, together with "a fair day's work for a fair day's pay", if coupled with a definite policy of refunding the international loans, will start things going briskly.

* Vice-Pres., and Gen. Mgr., Simmons Hardware Co., St. Louis, Mo.

Whether it is realized or not, the people are drifting rapidly toward Government ownership, and although the Nation plainly rejected the Plumb Plan and the vast majority of the people are against it, it may be adopted, unless the credit of the railroads is protected and any short-sighted action prevented which in effect would produce Government ownership and operation of the railroads.

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TRANSACTIONS

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Paper No. 1499

HIGHWAY TRANSPORTATION*

A SYMPOSIUM

By Messrs. Thomas H. MacDonald, H. E. Hilts, William G. B. Thompson, John N. Cole, E. A. St. John, Edward C. Lunt, Harry Meixell, Frederick Stuart Greene, Herbert S. Sisson, R. S. Parsons, and G. Wythe Munford.

With Discussion by Messrs. T. Hugh Boorman, Samuel Whinery, John C. Trautwine, Jr., Eugene W. Stern, Henry Goldmark, H. W. Brown, M. S. Ketchum, W. K. Hatt, and Edward W. Bush.

* Presented at the meetings of January 20th, 1922.

HIGHWAY TRANSPORTATION

By Thomas H. MacDonald,* Esq.

A recent study on the State highways of California made by the United States Bureau of Public Roads, discloses the average week-day traffic units to be divided in the ratio of 97.3% motor-driven and 2.7% horse-drawn. This result is quoted to indicate the degree to which, as rural pavement mileage is completed on the principal traffic lines of the State, highway transportation is measured in terms of the motor-driven vehicle. An adequate background, therefore, against which to project the discussions on highway transportation, is furnished by a brief résumé of the growth in the number of the major traffic units, motor vehicles, and the development of the roadways over which to operate these units. The proper combination of these two provides highway transportation, in a physical sense, characterized by the ease and speed with which it may be expanded and by the flexibility with which it may be adapted to a wide range of utility.

Table 1 is a comparative statement covering the eleven-year period, 1910 to 1921, which indicates the relative development of these component parts.

TABLE 1.

	1910.	1921.
Automobiles	487 000	8 404 000
Trucks	14 000	1 346 000
Totals	501 000	9 750 000
Maintenance fund	$ 25 000 000	$ 180 000 000
Construction fund	95 000 000	420 000 000
Totals	$ 120 000 000	$ 600 000 000

It is evident that highway development has lagged far behind the development of motor vehicles. This may be visualized strikingly by a comparison of the relative automobile registration and highway expenditures by years. The conclusions reached are:

1.—During the period, 1920-21, the potential number of motor vehicles demanding highway service increased more than 1 800%; while actual expenditures for highways increased about 400 per cent.

2.—During the period, 1910-18, motor vehicles increased more than 1 100%, and highway expenditures about 140 per cent.

3.—During the period, 1918-21, motor vehicles increased about 700% and highway expenditures about 260% of the 1910 figures.

In fact, it was not until the Federal Aid Act of 1916 that the public as a whole undertook to support the building of highways on an adequate scale compared with the already existing users of the highways.

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Since 1918 a large annual program of highway improvement has been carried on, and there is now the assurance of systematized development, beginning with the more important roads connected into a State and inter-State network, including not more than 7% of the total road mileage within each State. The total length of this proposed program will undoubtedly exceed 150 000 miles. The total mileage undertaken by the joint co-operation of the States and the Federal Government, on December 31st, 1921, had reached 35 500 miles, including all projects from those in the initial stage only, to those fully completed. This is the cumulative program of Federal Aid highways for the five-year period, 1916-21, which includes the period of the World War. In the last six months of 1921, 7 533 miles of the various types of highways were completed. The estimated expenditure for 1921 of $600 000 000 was derived from the following sources:

Motor vehicle licenses	19 per cent.
State road bonds	7 " "
Local road bonds	33 " "
State taxes and appropriations	12 " "
Federal aid	14 " "
County, township, and district taxes, appropriations, and assessments	14 " "

This total expenditure was divided, approximately, $420 000 000 to construction and $180 000 000 to maintenance. More than 50% of the construction program was carried on under Federal Aid projects, bringing the control under both the State and the Federal highway departments, and, in addition, a considerable part of the remaining construction fund was handled under the direct supervision of the State, thus bringing probably about 80% of the total construction expenditure under the engineering control of centralized departments. This is contrasted with 27% of the total expenditure in 1916, which was under the control of the State highway departments. In the twenty-six years prior to 1916, a gain of about 1% per year, in securing control of the highway funds, was made by the State highway departments. Since 1916, the increase alone has amounted to more than 50% of the total expenditures. The Massachusetts Highway Department, in 1893, was the first highway organization to engage in administrative and engineering control of the design, construction, and expenditures made by the State for building highways. When the Federal Aid Act of 1916 was passed, about thirty-two of the States had established highway departments, having varying degrees of control and varying amounts of highway funds with which to work. In 1916 and 1917, however, in response to the demands of the Federal Aid Act, seventeen new departments were established and many others were strengthened and given larger funds. As a consequence, all the States are now carrying on their major work under the direct supervision of State highway departments, and the supervision of the Federal Bureau of Public Roads extends in detail to all Federal Aid roads.

The typical State highway department is organized in two major divisions: Administrative and engineering. Quite generally the administrative heads are commissions, of which the members serve only part time and are responsible

for the major policies only. This leaves the entire executive and engineering control to the engineering staff, under a chief engineer or commissioner. Thus, the highway engineering organizations have been developed with definite administrative functions as well as with those of construction and maintenance. In the average department, the engineering organization engaged on the actual building of highways is divided into sections of design, construction, and tests. In the order of their authority, is the chief engineer, district or division engineer, resident engineer, and inspector. An estimate of the number of engineers employed by the State highway departments in 1919 totaled approximately 10 000. The construction program has practically doubled since then, but the number of engineers has not increased in the same proportion. On these men depends the character of the construction which goes into the highways.

Inspection is broadly interpreted as the engineering control that must be exercised over the construction program to insure first-class results. The building of roads must be considered as a manufacturing process, undertaken where the roads are to be built, not where this manufacturing process can be most advantageously performed. The wide distribution of the contracts undertaken annually within the average State, and the conditions under which the work is carried on, demands the highest type of engineers. The closest estimates that could be made in 1919, indicated that not less than 1 000 to 1 200 new engineers would be required in the highway work each year. These men must be furnished by the engineering schools, and with the opening of the construction season, the road-building industry will be able to absorb and will need more than are available of those fitted to assist in producing the character of roads needed. As indicative of the training of men needed for the highway program, and the character of instruction which should be received in the colleges and universities, the following are suggestions:

1.—Human relations: The ideals of public service; proper contacts with the public; and proper relations with the contractor.

2.—Management: Laws—State and national; layout of plans; material supplies; transportation; estimates; contractors' obligations; and engineering and administrative reports.

3.—Construction proper: Design—Interpretation and relative values; specification interpretations; and workmanship.

4.—Materials: Interpretation of laboratory tests; field tests; and soils.

5.—Traffic management during construction.

6.—Relative values: Economics of highway transportation.

These suggestions are not to be interpreted as advocating the adoption in college and university courses of superficial specialized instruction, but the curricula of the average engineering course should be and can be revised and broadened to give thorough training in some of the broad fundamentals of science, English, economics, and ideals of public service, in addition to sound courses offered in the Junior and Senior years, which will fit men to enter highway work much better prepared than they are at present. This responsibility, however, should not fall on the schools alone. The highway depart-

ments—State and Federal—must assume responsibility for bringing to the assistance of those in charge of engineering courses, contacts and opportunities of which those interested may avail themselves and thus obtain a correlation of theory and the practical application before graduation. The development of the short course for those already engaged in highway building, and of the graduate courses for research in special fields, are both of great importance in the advancement of engineering training and knowledge so seriously needed in the highway field. The close correlation and co-operation of the highway departments with the educational institutions are greatly needed, if the public is to be well served.

HIGHWAY INSPECTION

By H. E. Hilts,* Assoc. M. Am. Soc. C. E.

It has been found in the Pennsylvania State Highway Department that, in connection with the construction of a large mileage of durable highways, the work of the executive is that of an adjudicator and that the work at headquarters is simplified and systematized by the careful manner in which inspection work is handled on the construction contract. Mr. MacDonald has expressed the thought that inspection is not just inspection on contract work, and the speaker believes that all will agree that his views are conservative; nevertheless, the inspector can many times reach decisions that will lessen the work of both the district office and the main office of a department. The design of highways is subject to standardization and work of this character can be systematized. Plans also can be carefully checked, but inspection depends on the man assigned to it; therefore, all inspectors should be selected with care.

The work of a survey corps can also be systematized and checked, but after the work of an inspector has been done, checks may be made, but errors are costly to correct. In Pennsylvania, candidates for inspection work are not selected by Civil Service, the policy of "hiring and firing" being preferred. It is believed that all inspectors should be interviewed at headquarters before being employed. The candidate, on being employed, is told that he should consider himself a servant of the public, and that as far as emoluments of office are concerned, he must also realize that the satisfaction of building a road that is a credit to the Department and to the Commonwealth, must serve, in his own mind, as additional recompense. He is told that as far as the Department is concerned, he is not merely building Route Number so and so, but in the future the contract on which he is employed will be known as the road that Inspector "Smith" supervised. In other words, an attempt is made to appeal to the human element and arouse the candidate's ambition so that he will deliver a real job.

The inspection of construction is only a small part of inspection work. Preliminary to the making of surveys for the preparation of plans, the district engineers inspect existing roads, which they know will be improved in after years. In other words, every trip a district engineer makes over an unimproved road permits him to visualize re-locations or changes in roads, and in the spring of the year these trips are particularly valuable in the inspection of drainage conditions.

All construction plans are premised on careful studies of drainage, and an endeavor is made to locate both sub-surface and surface drainage structures so that they will answer not only the ordinary conditions, but also such extraordinary conditions as developed during the heavy winter of 1919, when roads that had been constructed for years, gave signs of distress during the spring thaw. These inspection trips, as they respect drainage, should be summarized

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by adequate notes which can be interpreted by the men in the office, who are designing the road.

During 1921, Pennsylvania constructed 678 miles of highway, largely made up of the so-called durable types—sheet asphalt or asphaltic concrete on cement concrete foundations, vitrified brick on cement concrete foundations, and one-course reinforced concrete. In expediting this program, the material situation was carefully studied during 1919 and 1920. Considerable time was spent during these two years, even during the construction of 253 miles in 1919 and 414 miles in 1920, in tabulating material supplies, both local and commercial, by counties and by States adjacent to Pennsylvania. The inspection of roads by district engineers is carried to a study of grade-crossing elimination prior to the issuance of the survey authorization.

After a survey has been made on the line established by the district engineer, a preliminary inspection with the blue prints of the project at hand, is made by the district engineer and a representative from headquarters, and if the project is a particularly difficult one, this preliminary inspection is made on foot. At this time, it is decided whether the alignment chosen is the best, and in checking this feature, it has been the endeavor to teach the men that the laying out of the highway, from the viewpoint of economy, must be developed by a study of distance, of maximum and ruling grades that will set traffic conditions, the elimination of unnecessary rise and fall, and of dangerous curvature. This does not mean that in highway work, the refinement used by the railroad engineer has been reached, because in many cases one cannot visualize or even approximately determine which of the previously mentioned features of construction is the most important, whether it is elimination of rise or fall, or curvature, maximum or ruling grades, or a study toward the elimination of distance. The traffic, as Mr. MacDonald has pointed out, has changed so rapidly in the last ten years that it would appear that even the present-day engineer would have to be the seventh son of a prophet in order to anticipate traffic conditions ten years hence. The checking of judgment by figures in establishing maximum grades, rise and fall, and curvature, which really amounts to the setting of business limits for economical expenditure of funds at this time, must be premised on anticipated traffic which the road will have to bear five, ten, or twenty years hence.

Another type of inspection is that of determining the kind of surfacing material which is to be used, and it is felt that, in deciding this question, an inspection on the ground by the district office is most important, so that full data may be secured as to the availability of materials, the nature of the termini of the road, and the present and the anticipated traffic which the road will take.

It has been found that the selection of type should not be based on the first cost alone, but should represent an investment, particularly where bond-issue moneys are concerned, which will, in some measure, be available after the bonds have matured, this investment not only being made up of grading and drainage structures, but also of the surfacing material. The competition between types, where such competition is based on engineering judg-

ment, is healthy and stimulates or results in lower prices, induced by more efficient business methods in the construction of the road.

The preliminary inspection as carried on by the district engineer is made almost simultaneously with an inspection made by the engineer of tests or his representatives, who examine the availability of local construction materials, the matter of inspection many times requiring from 3 to 5 days on a 3 or 4-mile project. The representatives of the engineer of tests blast out ledges and dig test pits, so that representative specimens may be obtained, and the quantity of materials available may be determined. This work has proven to be of extreme importance. In 1919, there were only about 8% of the contracts on which local material was used. In 1920, some local material was used on 28% of the contracts, and, in 1921, there has been an increase in the use of suitable local materials, which has necessarily resulted in a lowering of the first cost of the road.

Mr. MacDonald has pointed out the necessity for properly interpreting test results. The mechanical testing of materials, in a well ordered department, is mostly by systematized routine, and the speaker feels justified in stating that the most important feature of tests is their interpretation. Experience in this work is a fundamental requirement. The speaker might illustrate by a comparison of the different kinds of stone found in Pennsylvania, which vary from coarse-grained sandstones to heavy dense trap rock. There is no sand in many parts of the State, and it has been necessary to prepare sand artificially by grinding coarse and fine-grained sandstones. Studies were made, which pointed to justification in using artificial sands and in using good sands in combination with relatively poor aggregates, these results having been reached only after the construction of a test road in which various types of materials were used in combination. Test specimens were studied during the construction of the road and compared with specimens taken from the road after it had been in service for a period of six months.

The speaker has dealt with inspections made preliminary to surveys and during the preparation of plans. All these inspections have one objective, that is, to have the plans as nearly perfect and practical as possible, and to submit them to the contractor so that he may have full information regarding the roadway under consideration. Each contractor is given, on his proposal form, the estimated quantities as nearly as they can be determined and on a supplementary sheet he is furnished with the local materials which might be used in the construction work, and their location, with the provision, however, that no guaranty is given as to the continuation of quantity or quality of materials as sampled. It is found that the engineers of the department have used judgment in the preparation of all this material, but it is also suggested and recommended that the contractor, before basing his bids on local materials, should analyze the location on the ground and examine the test pits and the ledges.

The final field inspection also furnishes information on which to base the time limit on the contract. A 5-mile contract has been built by an efficient contracting organization, in as short a time as 70 days; another con-

tractor not as efficient might have required 3 years in building the same project. It is endeavored, as far as possible by inspection in the field, to work out a conservative time limit for every contract.

Only the engineering phase of contract work has been considered in all the inspection details mentioned. Another phase, however, is the inspection of the business of awarding contracts. This has been accomplished in Pennsylvania by calling to Harrisburg, for interview, each contractor who has been a low bidder. At this interview the Assistant Commissioner of the Department discusses with him the matter of equipment which he contemplates using on the work, this discussion being premised on a list of the major items of equipment which has been carefully prepared by the engineers of the Department. This list is prepared on the basis of the minimum amount that should be used in completing the work within the contract time, and it is thoroughly understood, by a gentlemen's agreement, that the equipment finally decided on at the conference will be used on the work. Furthermore, the contractor advises whether he owns the equipment, is going to buy new equipment, or is going to rent it. Careful attention is given, in the discussion, to the method of handling the work and as to the contractor's qualifications, based on previous experience in handling men, and as far as the labor situation is concerned, local or imported labor is discussed, and the establishment of commissaries is considered.

It has been difficult at times, during the past three years, to have materials delivered in sufficient quantities and with sufficient regularity to handle the construction work economically. Therefore, the Assistant Commissioner requires the contractors, during the discussion, to outline a fairly definite program, as to where they expect to purchase materials, so that the Department in turn from its previous experience can determine whether materials will be forthcoming, in order that the contractor will experience minimum delays. It has been found that this procedure is warranted, inasmuch as it may result in speeding up the work and in reducing the hazard of having to complete one season's contract during the following season, thus causing serious financial loss to the contractor as well as to the State, inasmuch as the new road will not be completed over the winter and spring months.

In order that contract work may be handled promptly and efficiently, every contractor must have financial resources and, in the interview with the Assistant Commissioner, he must furnish evidence that he is financially able to handle the work, not from a surety point of view, but rather from the point of view of ready cash or stocks or bonds that he can use to raise money, or, lastly, from the presentation of letters from banking concerns which express confidence in his ability and their willingness to finance him in an amount equal to at least 20% of the cost of the work. Following the usual practice, the contractor also furnishes a surety bond from an approved surety company in the amount of 50% of the contract cost. The interview with the Assistant Commissioner is not at all one-sided, inasmuch as he endeavors to impress on the contractors the thought that 100% work is being looked for and that plans and specifications are prepared to be followed, and, further,

that, in order to obtain a 100% job, the Department and the contractor must give each other 100% co-operation.

The inspection of materials is made at the source of supply, in order to protect the material man and the contractor, but this inspection is only preliminary. Inspection is also made at the delivery point of the contract, but this can be considered only as preliminary, the final inspection of materials being as they go into the work. It is endeavored to foster high ideals in the minds of the men by proper instructions in readable, illustrated form. The men have been instructed at headquarters and at district office schools, and three of the universities in Pennsylvania have established short courses in which particular emphasis has been given to the tenets of contract law and specifications.

Questions often have been raised as to the cost of making surveys, preparing plans, and inspecting contract work, and there is a lack of data in engineering literature on costs of this nature. Take about twenty-five contracts, which were available and on which tabulated costs have been completed; these jobs include those which have been finished rapidly and on which the plans were prepared quickly, as well as those which were studied for two or three years, and on which the contractor made slow progress, due to weather conditions, the non-delivery of materials, etc. It is found that the cost of these twenty-five contracts has averaged $524 per mile, for field and construction surveys; $175 per mile, for the preparation of plans; and $891 per mile, for inspection work; the total cost expressed in percentage of the contract price being 2.96. The cost of inspection and tests of materials has averaged about one-fourth of 1% of the contract price of the road. In other words, the engineering costs have averaged less than 3¼% of the contract price, and the speaker submits that this is cheap insurance to the taxpayers of the Commonwealth, and at the same time, furnishes the Department definite assurance that it has done its part in working out an efficient and economical method of handling contract work.

THE INSPECTION OF HIGHWAY CONSTRUCTION

By William G. B. Thompson,* M. Am. Soc. C. E.

Considering the importance of the duties and position of the inspector in the general scheme of highway improvement, there is a singular lack of information, or attempt at analysis of the reasons, for his existence. Few, if any, of the highway textbooks refer to the inspection of materials or workmanship, although they elaborately set forth the other factors deemed to be important to the proper conduct of any highway project, both as regards the quality of the materials and workmanship and the manner of their combining to secure the desired result.

The "Definition of Terms" found in most State highway specifications defines "Inspector" as follows:

"A representative of the Engineer authorized to make all necessary inspections of work performed and materials furnished under this contract. Such inspection may extend to all or any part of the work and to the preparation or manufacture of the materials to be used."

Further, "Duties of Inspector" are outlined as follows:

"The Inspector is authorized to inspect all work performed and materials furnished. He shall report to the Engineer the manner in which the work is being executed and the quantity and quality of materials received and used therein. He shall inspect all materials as received and, when there is any doubt regarding the quality of the same, notify the contractor to this effect. No materials shall be used until approved by him. When he discovers that any work performed, or material furnished, is defective, or does not comply with the requirements of these specifications, he shall notify the contractor and the Engineer to this effect. In case of any difference of opinion arising between the contractor and the Inspector as to the materials furnished, or the manner of performing the work, the Inspector shall have the authority to reject the materials and suspend the work until the question at issue can be referred to and decided by the Engineer.

"The Inspector, however, does not have the authority to increase or decrease the quantity of work to be performed, change the location, alignment, or grade of any of the items of the work, issue instructions that are contrary to the plans and specifications, or accept any material or work that does not comply in every respect with the requirements of these specifications. The Inspector shall in no case act as foreman nor perform other duties for the contractor, nor interfere with the management of the work by the latter."

In addition to these duties, many departments require inspectors to sign monthly estimates and the final statement of cost, without which the contractor cannot receive payment on the completion of any project.

From the preceding, one might deduce that the inspector has almost plenary power, yet, in actual practice, his power and authority are so circumscribed and limited that, on many occasions, even the most intelligent men are at a loss as to the proper course, and find themselves more or less of a nuisance in the eyes of the contractor; something to be endured possibly, and cajoled, but never, or rarely, to be taken seriously. That this attitude on the part of

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the contractor exists in hundreds of instances, no one familiar with the conduct of highway work will deny.

To what is this state of affairs due? Wherein lies its claim on the interest and attention of this Society, and what is the remedy?

It is due to various causes, many of which have their origin in the beginning of public works activities, and is a curious commentary on the frailty of human intentions, particularly as applied to the conduct of public improvements. One might reasonably expect that, given a clear plan of the work to be performed, a clear statement of the quality and quantity of materials to be used, and of the manner of their combining and use to effect a completed structure, the contractor could, and should, without inspection, deliver a completed structure complying with the requirements given him.

If such a method was ever tried, it evidently failed to meet expectations, and there resulted the appointment of an inspector, whose function was more that of a policeman or detective than that of an interpreter of the specification requirements. Thereafter, as public works increased in extent, the power of appointment to the position of inspector frequently became a political asset. He was placed on the work ostensibly to protect the public interest; actually, in many instances, he became the medium through which unscrupulous public officials and contractors mulcted the public treasury.

Instances are not lacking of shoemakers, saloon-keepers, and others, having no knowledge of, or experience in, construction work, being placed as inspectors to satisfy political debts or to build up and perpetuate a political machine.

Other causes, such as low pay, uncertain tenure of office, lack of opportunities for advancement, and the necessity of moving from place to place, have contributed to the existing lack of able, experienced inspectors, which, in turn, gives rise to the lack of mutual respect and harmony, too frequently found to exist between the contractor and inspector, although it must be admitted that conditions in this respect are far better than they were a few years ago, and there is a notable trend toward further improvement.

There is no other public activity in which engineers are interested, which compares, in extent and moneys involved, with the highway and street paving movement, nor will there be for many years. Neither is there any other public activity in which the destructive or deteriorative agencies follow so closely on the attempts of the engineer to build against them, as is the case of the highways and the heavily loaded, high-speed vehicles which use these highways and streets.

Research and testing are disclosing much, but the art of highway building has a long way to go before even the best specifications and the strictest observance of their requirements will deliver the road, proof against the assaults of the trucks. Is it not, then, of paramount importance that the most rigid compliance is secured with the specifications, requirements which present limited knowledge enables engineers to prepare; and in no way can this compliance be insured, except through the inspector who is the point of direct contact between the chief engineer and the contractor, and who is directly responsible, from the public viewpoint, for the integrity of the finished product.

The Panama Canal was the largest public enterprise prior to 1916. The Government was 10 years in finishing it, at a cost of $350 000 000 or

$375 000 000. At the present rate, $5 000 000 000 or $6 000 000 000 will be spent during the next 10 years for highway and street construction and paving, and highway bridges; and as the plans, specifications, materials, etc., are more or less fixed quantities, the unknown quantity is the inspector, since he, primarily, is responsible.

This does not assume that all contractors are dishonest, *per se,* or indifferent. Most contractors are honest and have a fine pride in their work, but the great amounts of money to be spent, attracts, along with the honest men, others to whom this work looks like "easy money".

How is the condition to be improved or remedied?

First.—As citizens, engineers should endeavor in their several communities to eliminate that curse on public works, political appointment made without regard to ability, fitness, or integrity.

Second.—Through its members who are highway engineers, or who are engaged in highway work, the Society should exert the utmost influence against the appointment of inspectors, except for ability, and by training, example, and precept, should raise the standards and ideals of their men. Some highway departments now have winter schools for their men, which are attended by inspectors during the slack season for one or two weeks, while under pay. These are good, and their establishment should be encouraged.

Third.—Provision should be made for the retention of inspectors at full pay through the slack months. If they are worth having during the construction season, they are worth keeping and paying through the winter. Much more is eventually lost to the public by engaging new, untried men every spring, than by paying an efficient corps for three or four months of the winter, and acquainting it with the results of research and knowledge gained during the year.

Fourth.—The importance of the inspector's position should be generally recognized; and his compensation and employment made as attractive as possible.

Fifth.—Assuming the existence and maintenance of a corps of trained, capable, engineering inspectors, the status and authority of the inspector's position should be more clearly defined, and instead of limiting his authority, it should be extended to permit of his making prompt decisions on the ground, thus eliminating the delay and financial loss to the contractor, which now frequently results when the engineer is unable to reach the job for several hours after any dispute arises.

Sixth.—It should be generally recognized that material producers frequently ship materials which do not comply with the specifications, thus causing delay and conflict between the contractor and the construction inspector. Differences between contractor and job inspector, and consequent delay and antagonism due to this cause, can be eliminated entirely by placing competent inspectors at points of material production to inspect and approve all shipments before they are billed. This method, if consistently followed, will eliminate one fruitful cause of conflict.

Seventh.—Many of the highway inspectors are engineering graduates, and, as such, are potential members of the Society.

FINANCING AND BONDING HIGHWAYS

By John N. Cole,* Esq.

Vital as transportation has been in the development of every nation, it has reached its highest place of importance in the development of the United States.

Transportation has followed along many paths of development, and it would seem as if out of all the experiences in the transition from the Pilgrim methods of intercommunication to the present types, something should have been learned as to the relation which each period should bear to the other. This is peculiarly true as to the problem of the wisest method to be adopted in undertaking the development of the new form of transportation as it now calls for consideration. Perhaps with no phase of such development have the American people been willing to profit less by past experience than in the particular problem of wise financial handling of the present-day situation.

The earliest transportation systems in Massachusetts, which perhaps may properly be called the virgin soil of America, required little capital outlay with such development work as was undertaken. When the first highways were undertaken, the private owner cut the trail and built the ford which afterward was replaced by the bridge over which he alone or with his associates, for a long time, exacted toll—all again without any reference to the problem of how the developed system of transportation should bear on the general community.

The more intensively developed agencies for transportation that have followed from that day until 15 to 20 years ago, gave hardly less heed to the future than these earliest agents, because the constantly developing new territory, and the rapid unfolding of the broader field to be served, afforded what appeared at that time to be unlimited opportunity for issuing securities on money invested and against labor performed, sometimes real and too often imaginary, on which dividends or interest charges could be easily placed and with little difficulty paid under a real or assumed earning, but too often produced out of new capital provided.

Then came the stock taking. The public no longer absorbed new securities for railroads, or undertook development of street railway lines having inception only in the fertile brain of the agile promoter. The development of the automobile caused the student of transportation securities to become seriously concerned over not only the problem as it then existed, but the future conditions that should control the work of placing the greatest factor in National civilization—its transportation interests—on firm and substantial ground.

From that time until the present, transportation facilities have been in a chaotic state which has been made more so by the demand for improved highways. The motor vehicle is a return to individual service, and because it is an individual service it is the popular service.

As long as highways were called on to bear a small load, hardly ever exceeding a ton, and at a slow speed, with freight almost a negligible quantity,

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the problem of constructing a surface that would carry fairly well the load, was not difficult. Also, as long as the general public was satisfied that, in the spring, the coming out of the frost might leave a highway unfit for travel, it was still less difficult.

The motor vehicle has changed all this, and pages might be written to tell of how it has been changed, without having anything presented more effective than to call attention to a result of the traffic census in Massachusetts, which has been taken biennially for many years. Selecting the same forty-four stations for a traffic census to show the movement and the development of freight per hour during the last twelve years, the figures in Table 2 are given.

TABLE 2.—Traffic by Weight, in Tons per Hour.

Year.	Horse.	Automobiles.	Auto trucks.	Total.
1909.....	200	160	....	360
1915.....	140	840	180	1 140
1921.....	70	3 500	1 260	5 530

Could any statement present more clearly the changed conditions under which highways must be constructed to care for the modern demands for motor transportation, unless a further statement is presented to show the tremendous increase in the public dependency on motor transportation, as given by the registration figures of motor vehicles in Massachusetts for use on January 1st, 1922, as compared with January 1st, 1921? Massachusetts compels every motorist who wishes to operate his vehicle on January 1st to be registered and carry his new number plates on that day. This rule has been strictly enforced for three years. Acting under it, there were registered for use on January 1st, 1921, less than 80 000 vehicles, and on January 1st, 1922, there were more than 140 000 vehicles. In 1921, before the close of the year, the 80 000 vehicles in Massachusetts had increased in number to 350 000. In 1922, it is estimated that the 350 000 of 1921 will have reached 420 000, a natural increase of 20% for the year, but the figures for January 1st show an increase of almost 100% in dependency on the motor vehicle for every day service. This enormous growth of the motor vehicle is duplicated in every section of the United States. On the part of owners and operators, there is a strong feeling that already motorists are contributing their full share to the problem of building and maintaining the highways on which they must operate to give their full measure of service, but the figures show that it is their job.

The Nation becomes each day more dependent on this particular type of transportation. The construction of modern highways is failing to keep pace with the demands. The competition between communities is more than a community pride; in many cases, it is a struggle for the existence of the community now entirely dependent on motor transportation. The speaker could cite case after case in Massachusetts where the only way out for residents and business is that provided by the modern highway and the motor vehicle. To meet this situation is the task. How to pay the bill is the problem. It is a problem that cannot be freed from its relation to the cost of all previous

systems on which there has been a capital outlay and it is vitally related to the cost of all future systems.

As a problem that cannot be freed in its consideration from the cost of previous systems, it requires careful study from the standpoint of how much destruction of previous capital outlay is justified in connection with expenditure for this new system.

Duplication of service is always uneconomic. The substitution of a more efficient form of service for one that has become inefficient, is always wise.

It is not new for the development of one type of transportation to destroy the investment in another. An example of fixing railroad rates so that long established and efficient waterway transportation has been eliminated, even though naturally a better service and more economically conducted, should lead us to go slowly in allowing this new system of motor transportation to bring about the same result in connection with either the developed waterway service or the long established railroad service, for with both of these systems it must be wisely co-ordinated.

Much more important, however, than this phase of the situation is the enormous dependence that National life and world life must continue to place on railroad service. A sound burden for the public to carry frequently calls for continuing several types of transportation if full service is to be secured, and the need of destroying old capital investment at the expense of the public must be recognized as a condition that must be frequently faced in changing from the old, unsatisfactory and uneconomic service to the modern system of motor operation. If this means a double burden, it frequently spells a complete service.

This brings up the second fundamental problem in relation to the cost and how that cost should be borne in connection with a future system. It is clear that if, at the outset, the public pays in a direct charge for motor transportation and in an indirect burden which an abandoned system still places on the public, because bonds have not been paid and because investment still is there, the original load has practically been doubled.

Another phase of the future, that must never be lost sight of, is the constant increasing cost placed on the public at large, for investigation, study, and research for a new system yet unknown, but sure to come, for "the old order constantly changeth".

Practically every highway in the United States in existence fifteen years ago made up a possession of the people of a value impossible to compute. In Massachusetts, the 23 000 miles of highway represent a property investment exceeding $20 000 000 in value. This came to the people in 1905 unincumbered, capable of further development, ready for such improvement as has since taken place. In meeting the needs for better roads, the nation has spent in that 15-year period, not less than $1 000 000 000 per year, and to encourage the development of a system of through routes has undertaken a program of bond issues that represent a total debt in 1921 in excess of $1 000 000 000. More than that sum has been placed on cities and towns to pay, and all of it will come due from 10 to 50 years hence, with more and more added every year.

TABLE 3.—Bonding of Highways in the United States.

Issues Are for Periods Ranging from 10 to 50 Years.
Amount of Issue Under 20 Years, $114 790 000.
Amount of Issue 20 to 50 Years, $922 085 500.

State.	Date of issue.	Authorized.	Unconstitutional or deferred.
Alabama	1920	$25 000 000	$25 000 000
California	1910-19	37 000 000	.....
Colorado	.....	5 000 000	5 000 000
Connecticut	1907-13	10 000 000	.....
Florida	.....	20 000 000	20 000 000
Idaho	1906-20	2 705 000	.....
Illinois	1918	60 000 000	.....
Iowa	.....	100 000 000	100 000 000
Maine	1912-19	5 000 000	.....
Maryland	1908-20	24 470 000	.....
Massachusetts*	1894-1917	20 050 000	.....
Michigan	1919	50 000 000	.....
Minnesota	1920	75 000 000	.....
Missouri	1920	60 000 000	.....
Montana	.....	15 000 000	15 000 000
Nevada	1919	1 000 000	.....
New Hampshire	1909-13	1 300 000	.....
New Jersey	.....	20 000 000 (for tunnel)	.....
New Mexico	1920-21	2 000 000	.....
New York	1906-12	100 000 000	.....
North Carolina	1921	50 000 000	.....
Ohio	.....	50 000 000	50 000 000
Oklahoma	.....	50 000 000	50 000 000
Oregon	1917-19	30 500 000	.....
Pennsylvania	1918	50 000 000	.....
Rhode Island	1907-20	1 800 000	.....
South Dakota	1919	6 000 000	.....
Texas	.....	75 000 000	75 000 000
Utah	1911-21	5 260 000	.....
Washington	1911-20	30 190 000	30 000 000
West Virginia	1920	50 000 000	.....
Wyoming	1919-21	4 600 000	.....
	.....	$1 036 875 500†	$370 000 000

* No bonds have been issued for highway construction in Massachusetts since 1917, and all bonds issued since 1913 have been for the layout and development of roads in the sparsely settled sections of the State, to bring outlying towns into touch with the larger communities. There is outstanding at the present time a little less than $5 000 000.

† Entire issue for construction, with the exception of $100 000 000 in Western States, which covers construction and maintenance. Of these bonds, $417 500,000 was issued in 1919, 1920, and 1921.

From Table 3, it will be noted that hardly a State in the Union is free from this system of relying on future generations to pay for present expenditures. Each year will add to this enormous total, and unless the public compels itself to appreciate the seriousness of this load, and unless there is a determination on the part of each community to pay the bills of the present out of the revenue of the present, serious times are ahead. It must be recognized:

(*a*).—That, as yet, there is no such thing as a permanent highway.

(*b*).—That each generation brings forth new peculiar problems and operations.

(*c*).—That no demand in any community in 1922 is any less than that in 1921, even though the demand seemed to be answered in 1921.

Many of the bonds issued by the States represent burdens not to be relieved for 40 years. Not a highway in the United States completed to-day and paid

for out of such a bond issue will be in existence at that time. New lines of travel will then be established. Enormously increasing demands will have been created. Different types of service will be seen from every angle. The vision of the human mind is unable to furnish anything permanent enough to stand the test of a 40-year period.

The speaker may well take pride in the position that Massachusetts holds in connection with this problem. She was the first State to begin the development of the improved highway. Like most States that have followed, she yielded to the pressure for an expenditure of more money than any single year could provide. Little by little the amount grew, until a large sum had been placed against the credit of the State for this improvement.

In 1917, the end came. Since then no bonds have been issued and during the period between 1896 and the present, the bonds have been retired leaving the Commonwealth of Massachusetts a little less than $5 000 000 in debt for this type of public improvement.

Not only has the State gone on a "pay-as-you-go" policy for itself, but by statute it has made it impossible for any city or town to borrow, for more than five years, money for highway improvements, and for this five-year period no money can be borrowed unless the specifications for highway construction conform to the standard fixed and approved by the State Department of Public Works.

The situation is largely due to the increased revenue coming to Massachusetts, as it has practically to every State in the Union, from the registration of motor vehicles.

It has been recognized and is firmly believed that the motor vehicle has created the problem and the motor vehicle should pay the bill. The figures given in Table 2 relative to the movement of freight at forty-four different points on the same day in three different years, clearly demonstrate that the burden placed on the highway is insignificant as far as every other form of vehicle is concerned, except the motor vehicle. For motorists to refuse to pay, is, from a Governmental standpoint, unsound, and from the standpoint of the motor industry, most unwise. Again, the speaker would state that the financing of the modern highway is the motorist's job, to be done to-day for to-day.

The enormous problem of rebuilding and making available nearly 200 000 miles of important highway in the United States, has hardly begun. The billions of dollars that would be necessary to realize that dream must be provided, but should be expended only as they are available from year to year.

The American people can no longer afford to fool themselves in the belief that they are building for the future when they are building modern highways. The problem of to-day is for to-day's solution. In no line of public work is that more evident than in the construction of the service which the highway must render to this newest type of transportation created by the development of the motor vehicle.

HIGHWAY BONDING FROM THE VIEWPOINT OF THE SURETY COMPANY

By E. A. St. John,* Esq.

Surety companies are looking forward to a large volume of highway bonding. Many of the States have extensive programs, with the money available or in prospect. The National Government has appropriated $75 000 000, which will be used jointly with an equal amount appropriated by the several States. There should be an enormous increase in the highway contract bond business during the next few years.

These contracts are really financial guaranties: First, that the contract will be performed; and, second, that all claims in connection with it will be paid.

Surety bonds should be issued only to specially selected applicants. There is involved: First, the adequacy of price; second, ability to do the work; third, reasonable and practical specifications; fourth, fair supervision; fifth, financial responsibility of the contractor; and, sixth, moral honesty.

Specifications should be prepared free of ambiguities so that every condition will be readily understood. The contractor has no voice whatever in the specifications—the engineer is made the sole judge of their meaning. Anything misleading, therefore, should be avoided.

The contractor takes practically all the risk of unforeseen conditions, unforeseen physical hazards—the risk of strikes and increased wages, of securing the necessary materials and paying increased prices for them, and of overcoming all conditions—he must do the work!

The highway department, in the performance of its duty, wants the work done at as low a price as possible, consistent with good workmanship and material. The contractors submit their bids in competition, some with the one thought in mind, namely, to calculate close and get the job; others calculate high, with the chance, perhaps, that their bid might be low by accident, and this sometimes happens. Although the average bid probably represents the fair price for a given job, it is not the average bidder who gets the work, but the lowest bidder. Not infrequently at the opening of bids a surety company will find itself on the bond of a bidder whose figures are so much lower than the average that no other company will accept any part of the risk as reinsurance. This causes considerable uneasiness, particularly if an underwriting point had been strained when the bid bond was authorized.

Barring the rare cases where a dishonest contractor robs the job and absconds or uses the money for some outside venture, the safety of the surety company is purely a question of whether the contractor can complete his contract and pay his bills, whether he can do it within the contract price, and, if not, whether his own resources will hold out.

The surety company is interested in seeing that the contractor gets a fair contract, that the specifications are just and reasonable, that the price is fair,

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that he is given a decent and fair supervision by the engineer, and that he gets fair treatment in the interpretation of the specifications.

Contracting is a hazardous business; therefore, efficiency is imperative. Good management wins; the skill to calculate the cost of a job accurately, the foresight to make contracts for materials with responsible concerns, and the ability to buy at the lowest prices.

Contractors and, indeed, engineers sometimes feel that the surety companies do not give enough weight to experience and ability, but rely too exclusively on the contractor's financial statement. The basis for underwriting is this: Give a clean record to a contractor who has requisite ability, the necessary plant, organization, and equipment. With no extraordinary variance in the bids, he must also have a margin of available cash, or its equivalent, not only to finish the work, but to take care of the ever-present danger of loss from unforeseen contingencies. This margin is about 10% on simple non-hazardous contracts, and goes to 20% on the extra-hazardous classes of work. This is the requirement when conditions are favorable. If the bids are out of line, if money must be expended for plant, if the contractor is without particular experience in this line, or has no record for efficiency in the handling of his work, a higher ratio of financial resources is required.

The underwriting of the particularly large volume of highway business in prospect is not as free from difficulty as some contractors and underwriters are prone to think. Contractors, hungry for work, are calculating close, apparently anticipating still lower labor and material prices, or willing to take a small margin of profit to keep their organization employed. This is dangerous, not only for the contractors, but for the surety companies as well, because when this tremendous program gets under way, there is likely to be a reaction upward in prices for both labor and materials; such a reaction would accord with normal economic experience. There is, of course, no likelihood of repeating the great crisis reached during the 1917-19 period, but there is likely to be at least enough reaction to wipe out the small margin of profit that some contractors are now calculating. This would reduce tremendously the margin of safety for the surety. If luck goes slightly against the close calculator who went after the contract in order to keep his organization employed, he may have difficulty in making both ends meet, and if it goes badly against him, the surety company may have to complete the job and pay the bills.

A particularly hazardous feature is when maintenance guaranties are required, for, in that case, the continued financial responsibility of the contractor is involved, as well as his knowledge of the various kinds of surface pavements, and their suitability to climate and traffic.

No feature is of greater importance than that of availability of the contract price or the earned retained percentage as security for the surety where it is compelled to pay material claims incurred by the contractor, and this involves the financial relations of the contractor to the banks and to money lenders. Where the contractor, in order to borrow money, assigns the estimates and retained percentages to the lender, it has been held in some States

that the latter take these moneys in preference to the surety company that pays the material claims.

The contracting business is inherently hazardous. Some of the ablest men engaged in the business, after making big successes, have failed. The president of one of the prominent surety companies is reported to have said that few contractors or contracting firms have a record of more than twenty years—most of them fail within that time.

It is convenient for a contractor to be able to furnish a surety bond. It takes nothing out of his surplus, except the premium, enables him to recover his "good faith" check which accompanied his bid, and places him in a position where he can avail himself of all his assets and resources in the performance of his contract.

As compared with any other known practical method, it is economical to furnish a surety company bond. The alternative in New York State is a higher percentage of retention, and, also, the retention of the check deposited with the bid, until 15% of the contract is performed. Should 15% of the contract not be performed, the check is forfeited as liquidated damages. Without a bond, the increased percentage of retention applied to a contract of $100 000 works out about as follows: Assume for the purpose of discussion that a contract is performed in four equally productive months; of the $25 000 worth of work performed, the contractor would receive at the end of the first month $20 000, and about this time his $3 000 bidder's check (3% of the contract price) would be returned. To the time of the payment of the first estimate, therefore, the contractor would be deprived of the use of his $3 000 deposited with his bid, which would not be the case if he had given a bond. In addition to the $3 000 thus held, there would be $5 000 deducted from the first estimate, so that if the $3 000 were not considered at the moment of the payment of the estimate, which is likely to be the case, the State, at that time, would be holding $8 000 of the contractor's money, on a $100 000 contract, of which only 25% had been completed. Carry the illustration through the contract: Assuming the return of the $3 000 deposit check soon after the payment of the first estimate, the contractor would still have not less than $5 000 of his earnings of his first estimate held by the Highway Department, and an equal amount of $5 000 would be deducted from his second estimate, so that at the half-way point, the contractor would be deprived of $10 000 of his earnings. At the end of the third month, it would be $15 000, and when the contract is finished and he is still waiting for the final estimate and payment, he would be deprived of $15 000, plus the gross estimate for the last month, or $40 000 in all, having received from the State not more than $60 000 on a $100 000 contract.

Contrast this situation with the advantages of giving a surety company bond. When a bond is given, not only is the $3 000 deposit check made available for the contractor's use, but he is entitled to $90 000 of his estimates, which makes a material difference to him in the amount of money with which he is to operate, and may well mean the difference between success and failure in the performance of the contract, since many contractors fail by reason of money shortage. It can be seen that the payment of the small

premium required for a surety bond is only a trivial matter, compared with the greater advantages to the contractor in having the use of his money.

In regard to service in times of difficulty: Where corporate sureties are interested as sureties for contractors, it sometimes happens that by reason of the interest of the sureties, and the co-operation between the contractors and the sureties, the contractors derive a material advantage. This is well illustrated in the work which brought about a very equitable statute passed by the New York State Legislature and approved by the Governor in 1919, commonly referred to as the "Knight Bill". By reason of the unusual conditions arising from the declaration of war by the United States, the fulfillment of highway contracts by the contractors was made difficult and, in most cases, rendered impossible. The prices of labor and material soared to prohibitive heights, and neither was available in sufficient quantity or quality to perform the contracts then outstanding, some of which had not been started and others of which were in various stages of completion. There were, perhaps, 250 or 300 of these contracts, varying in amounts from $20 000 to several times that sum.

For the State to have attempted to enforce the contracts, without any relief whatever to the contractors, would probably have resulted in bankruptcy for 90% of the highway contractors in the State. The reaction would probably have resulted in long and tedious litigation of all these contracts or the greater part of them, paralyzing of the State highway work within the State, and practically depriving the State of the opportunity of availing itself of the skill and experience of a majority of these contractors, which, in itself, would have been a distinct loss. In that situation, the sureties took an active and, indeed, a leading part in gathering data, analyzing the situation, and presenting the problem to the Legislature and other officials in Albany, which resulted in the passage of the Knight Bill. This bill provided that the State should pay a dollar for a dollar's worth of work at the time of performance, which, in view of all the circumstances, was the only equitable and fair thing to do. It also provided for the cancellation, as of April 6th, 1917, the date of the declaration of war, of all highway contracts then outstanding and uncompleted, in which there had been no default other than the failure to complete; and to those contracts that had been partly performed in 1917 and subsequent to April 6th, and in 1918, it provided for reimbursement to the contractors of the excess cost of such work in 1917, not to exceed 35% of the price of labor, materials, and transportation, and for 1918, not to exceed 50% of the price of labor, materials, and transportation thereof, to be determined by an action and hearing before the Court of Claims.

It is doubtful whether the contractors, without the co-operation of the sureties, could have achieved this result. Although it cost the State some money to make provision for the increased cost and the re-letting, the State paid only what the work was worth at the time it was done, and probably paid less in the end than if the contractors had been forced to go through with their contracts as far as they could. This was an unusual situation, arising from circumstances over which neither the contractor nor the State had control, and which could not have been foreseen by either, but it

serves to illustrate the value of the service of the sureties in emergencies and under unusual circumstances. All three parties profited—the State, the contractor, and the sureties.

Let the matter be considered briefly from the standpoint of the surety companies. Some wonder has been expressed by contractors who are compelled to pay the high premium charges for contract bonds. If the underwriters of surety companies were endowed with the necessary wisdom to select contractors with unerring judgment, they could in practice, make the writing of bonds what it is in theory, namely, the rendering of a service—the selling of the use of the name and credit of the company for a specific purpose and for a specific price—that is what the surety companies strive to do, and if they could succeed, it would be necessary to charge only enough to pay operating expenses, plus a small margin of profit. In practice, it does not work out that way. The underwriters make mistakes. They bond contractors who fail, and, by the same token, they turn down contractors who finish the job. The companies select the best underwriters they can find and give them the benefit of all the statistics they have, and notwithstanding they have every incentive to separate accurately the "sheep from the goats", they make mistakes. The result is that the companies sustain large losses and find it necessary to charge a premium which will not only pay their operating expenses, plus a small margin of profit, but also pay the inevitable losses. It is to the interest of all concerned to keep these losses at a minimum—the lower the loss ratio the lower the rates. The average rate for construction bonds is 1½% on the contract price. It, therefore, takes 75 bonds to produce in premiums an amount equal to the average contract, but approximately one-half of the gross premium goes for various expenses; it, therefore, takes 150 separate risks to produce enough net premiums to pay one total loss. The odds, therefore (assuming the loss to be a total loss), against the underwriter, are 1 to 150. The average loss, however, is less than the full amount of the contract.

The surety companies are hopeful that the experience in the future will be better than it has been during the past; and if such experience justifies it, there can be no doubt that a reduction in rates will follow.

It is much easier to lay down rules than it is to apply them. In the first place, it is difficult to get accurate and complete information. Many financial statements are in round figures and difficult to verify. More or less dishonest contractors, and more or less dishonest brokers, have learned to deceive the underwriters of surety companies. Picture an agent of a surety company pilgrimaging to the capital of a State to be present at the road-contract letting, meeting the contractors there, ready to bid, escorting them to a local bank and arranging to give the contractor the necessary certified check to accompany his bid on the written agreement that should he prove to be the lowest bidder, the surety company would sign the final bond; then consider, further, the ludicrousness of a man without any experience whatever as a road contractor, without any financial responsibility, and, therefore, of course, without any plant, submitting a bid without going through the formality of making any figures, but, accepting the engineer's estimate as a reasonably safe figure, submitting it with the certified check arranged by the surety agent,

and, then, proving to be the lowest bidder! Up to this point, his plan had worked out. His thought was that one of the other contractors would buy the contract, yielding him a profit, but his bid was so low that no contractor would have any of it. That is the position in which a surety company found itself, and it provided a contractor to do the work with considerable loss to itself. Competition among the surety companies is almost as keen as it is among the contractors.

We have spoken about "busted" contractors. The mortality among surety companies has also been high. The United States Treasury Department, for years, has issued a quarterly bulletin showing the amount for which the respective companies can qualify on a given bond in favor of the United States Government. About as many companies are listed on this bulletin as there were a dozen years ago, but, to-day, of those original companies one will not find more than five listed, which means there are just five of them left. It is well known what caused the failures of these companies—it was principally poor contract underwriting. Something has been learned from these dead surety companies, but there is still much to learn.

FINANCING AND BONDING HIGHWAY WORK

By Edward C. Lunt,* Esq.

The speaker will try to show how surety companies underwrite highway bonds; that is, how it is determined whether or not they can prudently issue them. The first thing to be considered is the contractor's financial condition. If his resources are found to be slender compared with the size of the job, that fact seems almost fatal in itself, without regard to any other considerations.

Although underwriting methods in general are freely criticized by agents, contractors, engineers, and highway commissioners, this particular item in the underwriting creed, the insistence on financial responsibility, receives more than its share of disapproval. For example, the surety companies are naturally pleased when it is found that an applicant for a highway bond is not only a contractor of the usual type, but also an engineer; but even that favorable factor seems to them not to supersede the necessity of financial resources commensurate with the magnitude of the given job. Not all engineers are richly endowed in that respect, and sometimes these engineer-contractors rather resent the application of sordid considerations of this kind to their cases.

Surety companies regard a strong financial condition as a *sine qua non* of bondability. They like to feel reasonably certain, not only that the contractor will be able to finish the work by reason of his equipment, plant, experience, and general capacity, but also that he has sufficient financial resources to tide him over any of the numerous mischances to which his hazardous calling is subject. They like to feel confident that it will not mean loss to him and to them if, through some contingency unforeseen and perhaps unforeseeable, he finds his labor costs rising beyond all calculable forecasting, or if, at some critical stage of his contract, a freshet sweeps down the valley, ruining his half-completed work and turning his costly equipment into so much junk. Under such distressing conditions, nothing but solid surplus assets stand between the surety company and heavy loss.

If the unfortunate contractor is able to qualify for suretyship, as to this first requirement, the underwriter next considers the man's experience in the given line of work, his plant, equipment, etc. If the contractor has been engaged for years in highway construction and has been reasonably successful in such work, the surety companies are satisfied; but if it is found that he is just starting in business, they wonder whether his inexperience may not handicap him seriously and may not ultimately make them his partner or, perhaps, his successor in the given job. They fear that he will be like that other contractor, a bridge builder in that case, who found himself in desperate circumstances and knew that he must fail unless he could secure an extension of a large loan about to mature at his bank. When the president of the bank absolutely refused to renew the loan, the contractor asked him whether he had ever been in the bridge-building business. "No," he replied, "of course not." "Well," the contractor added, by way of breaking the news gently, "you

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are in it now." Similarly, underwriters sometimes find themselves plunged with equal abruptness into some mysterious and difficult business for which nothing in their previous experience has prepared them in the least.

A third underwriting point that is deemed of prime importance is the matter of other work on hand. If it is found that an applicant for a bond already has a number of big jobs under way, in various stages of completion, the underwriters are not over-enthusiastic about his case. Many contractors are too ambitious, they want to get rich too fast, and they undertake a great deal more than they can thoroughly accomplish.

Two or three points for which engineers are directly responsible have an important bearing on the decisions of surety companies. When engineers provide that the contractee shall be privileged to withhold a certain percentage of the contract price as the work progresses and after the completion thereof, as a protection against liens and undiscovered defects, they are incidentally benefiting the surety company, and the company is correspondingly grateful. Such a provision in the contract has often saved the company from loss. It is quite another story when, on the other hand, engineers include in their specifications penalties for delayed performance and maintenance guaranties. Underwriters regard maintenance provisions in highway contracts as altogether unfair to contractors. Performance and maintenance guaranties are reasonable, no doubt, in connection with the sale of machinery, patented devices, and the like; but they seem to be illogical and unjust to the contractor where the latter undertakes only to do a certain piece of work in exact conformity with detailed specifications laid down by the contractee. Under these conditions, responsibility for results would seem to rest logically with the framer of the specifications rather than with the contractor who merely carries out such specifications. When an engineer prescribes the precise manner in which a highway shall be built, it would seem to be no affair of the contractor whether the resultant highway stands up under the traffic for six months or six years. In the case of road contracts, maintenance provisions seem particularly unfair to the contractor both on general principles as indicated, and because a road designed for and normally subject to comparatively light traffic, from unexpected causes, may be forced to carry continuous heavy traffic during the maintenance period.

It may be questioned why the speaker has not included engineering considerations in his list of underwriting factors. There are good reasons for that. Because of the vast variety and number of bonds that must be disposed of with despatch, underwriters have no time, even if they had the ability, to consider them from an engineering point of view. In a single day, a contract underwriter will pass on one hundred or more important bonds—bonds guaranteeing the construction of a big office building, say, a huge coffer-dam, a concrete viaduct, a 30 000-ton steamer, and a union station somewhere, perhaps all intermingled with filtration plant, dredging, and tunneling contracts, and a dozen or more big highway awards. How could any underwriter possibly apply in a single day engineering tests and standards to a situation of that kind, except in the most superficial way? From the nature of the case, the great

bulk of the business must be underwritten from a banking, rather than from an engineering, point of view.

The speaker hopes that he has indicated the determination of the surety companies to follow conservative practices in respect to highway bond underwriting. They deem such a course best for the contractors, the highway department, and every one concerned, for numerous reasons and, especially, because it tends to keep the business in the hands of competent and responsible people. Every interested party is anxious to have the work let to responsible contractors, to have it done on time, and in rigid accordance with the specifications. In the speaker's opinion these vitally important ends are directly and efficiently promoted by careful and prudent underwriting on the part of the bonding companies.

THE MOTOR VEHICLE IN HIGHWAY FINANCING

By Harry Meixell,* Esq.

It is estimated that, during 1922, the United States Government, its States, the counties within the States, and the municipalities within the counties, combined, will spend fully $1 000 000 000 for highway purposes. These purposes include carrying and amortization charges on outstanding highway bonds; expenses for the administration of highway departments; maintenance of highways; construction of new highways; reconstruction of outworn highways; and the regulation of highway traffic.

Such a vast expenditure has been made necessary and desirable because of the insistent and justifiable demands of 10 000 000 motor vehicles for a wide-flung network of stronger and better highways, amalgamated into co-ordinating National, State, county, and local systems. More important than this is the fact that these vast expenditures have become necessary and desirable, because only through them can the maximum economic development of the country be achieved. No matter what one may think or say regarding the relative importance of the various mediums of transportation, highways are always the immediate stems on which farms, factories, stores, and homes hang as leaves, taking nourishment from and giving it to the whole tree of economic growth.

Notwithstanding the importance of highway transportation and the overwhelming part of the motor vehicle in it, law-makers have been too prone to take the position that highway improvement confers special, and as some would have it, exclusive benefits on the motor vehicle. In consequence, they insist that it should bear the entire burden of public highway financing. In other words, they would have the 10 000 000 motor vehicles now registered in the United States pay for the privilege of operating on the highways, the $1 000 000 000 which has been estimated will be spent for highway purposes in the United States during 1922.

Before discussing the merits or demerits of this contention let us examine briefly the registration fees, gasoline taxes, wheel taxes, and other forms of impositions which motor vehicles now pay into public treasuries, and, on these as a basis, seek to estimate what portion of the $1 000 000 000 highway program for 1922 the motor vehicle will finance. After that, an effort can be made to determine what portion it should pay.

The latest figures available on the subject are those which cover the year 1920,† as follows:

Federal:

1.—Passenger car excise taxes	$83 600 294
2.—Commercial vehicle excise taxes	15 160 456
3.—Parts, accessories, tires, excise taxes	49 960 128
Carried forward	$148 720 878

* Secy., Motor Vehicle Conference Committee, New York City.

† "Facts and Figures of the Automobile Industry", 1921 Edition, p. 48, pub. by the National Automobile Chamber of Commerce.

Brought forward		$148 720 878
State:		
1.—Registration and license fees	$102 000 000	
2.—Personal property taxes	50 000 000	
3.—Miscellaneous taxes (motor fuel taxes, motor transportation franchise taxes, mileage taxes, business taxes on manufacturers and dealers)	5 000 000	
		157 000 000
Municipal:		
1.—Registration and license fees	$1 000 000	
2.—Miscellaneous taxes	10 000 000	
		11 000 000
Grand total		$316 720 878

As $168 000 000 of the grand total of $316 720 000 for 1920 was yielded by State and municipal taxes on 9 211 295 motor vehicles, and as this registration was an increase of 22% over the number registered in 1919, it is fair and conservative to assume, despite the current period of economic depression, that by the close of 1922, the two-year interval will develop at least an increase of 25% in the number of registrations recorded at the close of 1920. On this assumption, and on the same scale of taxes and fees that obtained in 1920, one can logically expect State and municipal taxes, in 1922, to yield 25% more money than in 1920, or approximately $210 000 000.

This, however, does not tell the whole story. During 1921, forty-two State legislatures met in regular session and five in special session, and with few exceptions the law-makers of these States increased the annual registration fees to be effective for motor vehicles during 1922, from 10%, 15%, and 20% to as much as 400% and more in certain cases. Moreover, the tax rate on the motor vehicle as personal property has everywhere had an upward tendency, and to add to the impositions, eleven new States, making a total of fifteen at present, are taxing gasoline and other fuels used by motor vehicles.

During 1922, only eleven State legislatures will hold regular sessions. Nine of these have already convened and scarcely had their deliberations begun, before bills, providing for gasoline taxes, increased registration fees, wheel taxes, etc., for the motor vehicle, put in their appearance.

In view of the figures given, and in view of the circumstances mentioned, is it too much to believe that during 1922, the motor vehicle will pay $250 000 000 into State and municipal treasuries? On the other hand, lower production of motor vehicles and accessories in 1922 than in 1920 will decrease the Federal excise taxes to about $100 000 000, making the grand total for all impositions on the motor vehicle $350 000 000 for the year.

With the exception of the personal property taxes, these levies on the motor vehicle are special or exclusive. They are not imposed on farm machinery, for instance, in spite of the fact that the 3 000 000 to 4 000 000 motor vehicles owned and used by farmers are hardly less useful and necessary to them than their plows, harrows, cultivators, and other tools. Why should agricultural implements and countless other products of the factory be exempt from excise

taxes, and the motor vehicle and its parts be called on to pay $1 500 000 of such taxes annually?

As for the personal property tax, here, too, although this levy is presumably general in application to all kinds of personal property, it usually falls with special and even discriminatory force on the motor vehicle. As evidence of this, a typical case in a near-by State can be cited where the personal property tax levied on an automobile exceeded the same tax on all the household goods including pianos, talking machines, rugs, etc., in the house of the owner of that automobile. In effect, therefore, if not in intent, the personal property tax levied on motor vehicles in thirty-seven States is proving a special imposition.

Indeed, when all the taxes imposed on motor vehicles by the Federal Government, the States, counties, and municipalities are closely examined, it can be seen that either in theory or practical application they bear with exclusive or special force on the motor vehicle.

This situation makes it apparent that even if it were to be admitted that highways are of special benefit to motor vehicles, these same motor vehicles, during 1922, will contribute in the form of special fees, thirty-five cents for every dollar that will go for public highway financing in 1922. Will this highway financing, however, specially or exclusively benefit the motor vehicle? The answer to this question, coupled with a quantitative and qualitative estimate of these benefits, alone can enable a determination of what part of the highway financing the motor vehicle should pay.

Who Benefits from the Highway?—A man living on the outskirts of a city is suddenly stricken with a serious illness. His doctor speeds to his side in an automobile, diagnoses his malady, prescribes an operation, rushes him to a hospital in an automobile ambulance, and by virtue of the time saved by motor vehicles operated over good highways, saves the man's life. Who used the highways? Who benefited from them? Obviously, both the sick man and the doctor, but where can the line be drawn? Should the sick man who owned no automobile, but who was made well by a combination of motor vehicles and good highways, pay nothing or everything or 25% toward highway financing? What should the hospital and the doctor who owned and operated the motor vehicles, and whose efficiency were increased thereby, pay? Clearly, the mere putting of these questions demonstrates the futility of seeking an accurate answer by this method of assigning intangible and unmeasurable benefits.

Let us try another example: By virtue of motor vehicles and good highways many farmers, factories, jobbers, and retailers are now avoiding heavy handling, transportation, and marketing charges, thereby greatly increasing their gains, but, at the same time, lowering costs to the ultimate consumers, many of whom do not own automobiles. Who are the beneficiaries in this case? What are the kind and degree of the benefits? How should the costs of the highways which made the benefits possible be apportioned?

Illustrations of this kind could be cited indefinitely, all tending to show that the advantages of good highways accrue alike to the public as a whole and that part of the public which owns and operates motor vehicles. In this case, again, however, such illustrations prove how impossible it is to take bene-

fits, which are shared by the general public and the motor vehicle owner alike, as the measuring stick for the relative portions of highway financing that both should carry; and do not forget that the motor-vehicle owner is also a part of the general public, as is the non-owner, and does not escape any of the general public's burdens by virtue of his ownership of a motor vehicle.

There is a beneficiary who derives untold advantage from the motor vehicle and good highways, yet in all discussions of the subject and virtually in all legislation he is entirely overlooked. He is the owner of real estate along the improved highways. When the owner of 50 acres of land, on an unimproved dirt road, will accept $100 per acre for that land prior to the conversion of the dirt road into an improved thoroughfare, usable at all seasons of the year, and following the improvement refuses an offer of $200 per acre, it is obvious that he has received a direct, immediate, and tangible benefit of more than $5 000 by the mere completion of the public improvement. Furthermore, he benefits indirectly by having his land placed in economical contact with all other territory that the highway and its connections serve, and this circumstance, in turn, as the flow of traffic increases, advances real estate values, ever thus accruing to his direct benefits.

The Fundamentals of Highway Financing.—Leaving the general public and the owners of property abutting on improved highways to fight out, what, if any, shall be their relative shares in highway financing, let us seek to determine what, in sound economics and justice, the motor vehicle should pay toward the vast annual outlays that the Federal Government, the States, counties, and municipalities are making for good roads.

An intelligent decision regarding this proposition can only be reached by a realization of the vital elements of highway financing. Careful analysis shows that these fundamentally consist of capital investments and current expenditures. Under the former, should be included the cost of surveys for highways; acquirement and clearing of rights of way; grading; drainage; laying of foundations; original construction of highway pavement; widening of highways or pavements; and reconstruction after a highway can no longer be economically maintained. Furthermore, where the capital investment has been made through the deferred payment or bond issue method rather than by the pay-as-you-go or cash method, the carrying and amortization costs for such bonds should be charged to the capital investment account. On the other hand, current expenditures should embrace the maintenance of highways, regulation of their use, and administration of the highway departments.

After giving serious study to these matters, the Motor Vehicle Conference Committee, composed of representatives of the American Automobile Association, Motor and Accessory Manufacturers' Association, National Automobile Chamber of Commerce, National Automobile Dealers Association, Rubber Association of America, and the Trailer Manufacturers Association of America, has definitely taken the stand, that the aggregate amount of excise taxes and all those other forms of special impositions levied on the motor vehicle, should in no case ever exceed the amount of money needed to meet these current expenses. Moreover, the Committee is insistent that maintenance charges should be levied on the motor vehicle only for highways

properly located to meet the economic needs of the community and built of such materials and in such a manner that they will efficiently and economically meet those needs. It is contended that the general public—which includes motor-vehicle owners as well as non-owners—and abutting property owners who are also made up of motor-vehicle owners and non-owners, should meet the cost of the capital investments necessary for improved highways.

When it is realized that the American public, prior to the era of motor vehicle transportation, spent scores of millions of dollars for the improvement of highways for animal-drawn transportation and never charged that transportation a penny of special taxes, either for capital outlays or current expenses, it can be seen that the position of the motor vehicle as reflected by the Conference Committee is fair, if not generous.

More than that, if the proposition must be accepted that good highways confer special benefits on the motor vehicle, then special motor-vehicle impositions for the current expenses necessary for maintenance, regulation, and departmental administration, are perhaps the most accurate measure that can be found of those special benefits. Motor-vehicle owners, as a substantial and heavy paying part of the great army of general taxpayers, together with non-owners, contribute toward the capital outlays necessary to establish improved highways that redound to the general good of every one. Then come the motor-vehicle owners to use these highways. Disregarding the fact that this use, more often than not, is advantageous to others beside these owners, it must be admitted that they subject the highways to wear and tear. The special fees which they pay, offset this wear and tear. Here, then, is a justification for exacting from the motor vehicle the current expense fundamental of highway financing. After this is granted, however, why should law-makers, with increasing insistence, call on the motor vehicle for greater taxes and extra taxes to meet capital outlays?

To be sure some may disagree with the particular phases of highway financing which have been listed in this discussion under the headings of capital outlays and current expenses; it is almost certain that there will be the greatest diversity of opinion as to what the expressions, highway maintenance, highway regulation, and highway administration should include. Even so, the speaker is confident that once those who are working on the problem shall have agreed on what shall constitute capital outlays and current expenses, they will subscribe to the position that special tax impositions on the motor vehicle, in Federal, State, county, or municipal highway financing, should never be greater than is necessary to meet current expenses.

FINANCING AND SURETY BONDING

By Frederick Stuart Greene,* M. Am. Soc. C. E.

The speaker was especially interested in the bill which allows a contractor, doing work in New York State, to bond himself. When it came before Governor Alfred E. Smith there was much opposition, and the speaker urged the Governor to sign this bill, largely because of information obtained immediately after his taking office as Commissioner of Highways.

The speaker was told that one of the best contractors in New York State, a man who had never failed to carry out every provision of his contracts and who had had years of experience, had been refused a bond by all the companies. This contractor was forced to deposit Liberty bonds before the contract could be awarded to him, in spite of the fact that he was financially sound.

The speaker was informed that the State of New York had paid indirectly, since it began the construction of highways, approximately $5 000 000 for surety bonds (that is, the contractors had paid this sum and the State, of course, had necessarily paid this additional amount for the contracts), and that the State had received less than $100 000 paid in by the surety companies to compensate it for losses due to broken-down contracts. It seemed that if this was the case, it would be wise to permit the contractors a chance to show what they could do toward bonding themselves.

For these reasons the speaker urged the Governor to sign the bill which allows the State to retain the contractor's bidding check, amounting to 3% of the contract, until the amount of the retained percentage equals that check, whereupon the check is returned to the contractor. The speaker is glad to state that it was possible to deposit these checks so that they might draw interest, and the interest was allowed to the contractor on the return of the check.

For the information of commissioners in other States, there were two faults in the New York bill: It allowed the contractor to elect whether or not he would give a surety bond. This option should be left with the Commissioner. One case arose during the speaker's term of office, where he was not willing to allow the contractor to proceed without a surety company's bond, and his only recourse was to declare that if the contractor did not put up a surety bond the Commission would throw out all bids and re-advertise.

The greatest objection brought against the bill of retaining 20% as a safeguard to the State was that it would encourage unbalanced bids. The contractor might bid a high price on excavation, for example, and a low price on pavement. After finishing the excavation at a high price, he might then abandon the contract, thus leaving the State to take care of itself. The solution of this was easy. The Department made a ruling that no proposal would be received in which the price for any item exceeded the engineer's estimate for that particular item more than 10 per cent. This placed the responsibility where it should be, namely, with the Department and with its engineers, to see that they did not have an unbalanced estimate which, in turn, would permit the contractor to submit an unbalanced bid.

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The experiment of permitting the contractors to bond themselves has been proven to the advantage of the State of New York. There is only one contract that the speaker can recall, by which the State sustained a loss, and this may prove to be only a probable loss. In all other cases, where the contractor has bonded himself, the plan has worked satisfactorily.

The speaker would like to suggest an improvement to the present law. The State now holds 20% until the actual completion of the contract. This might result in an undue hardship to the contractor. For instance, suppose a contractor has completed, in the late Fall, all his contract except some minor detail. The Commission could then hold 20% of all the work done, during the winter, because of the non-completion of only a few dollars' worth of work. In the State of New York there is a ruling which was carried on under the speaker's administration, and is still being carried on, which allows the contractor having only a small amount of work left to do, to file a bond or sufficient cash with the Commission to cover this work, and the contract is then accepted and paid for.

To obviate this fault in the law, the speaker believes it would be more equitable to the contractor if the law was amended somewhat as follows: 20% to be retained until 60% of the work is performed; 17% to be retained until 80% of the work is performed; 15% until 90% of the work is performed; and thereafter 10% until the entire work has been performed to the satisfaction of the State.

The speaker would estimate the cost of a bond to the contractor about as follows: Starting on the basis of a $100 000 contract: At 1½%, the bond would cost $1 500. Remember that, whether or not a bond is put up, the State will retain 10%, and if a bond is not put up 20% will be retained. It is fair then to calculate the interest charges on only $10 000. At the end of the contract, if no bond was given, there would be $20 000 held, but there would be $10 000 if a bond was given. Assume only the $10 000 extra that would be held by the Commission, on the no-bond plan, and that this $100 000 contract is completed in twelve months. It is fair then to assume that an average of $5 000 of that extra $10 000 is retained for six months; and $5 000 at 6% is $300, as against $1 500, the cost of the same contract with a bond. If a sliding scale, as suggested, should become a law, the no-bond contract would be still more favorable to the contractor.

The speaker has heard of a new firm starting to build highways, one of the members of which has been in the contracting and engineering business for thirty years. Although he had never made a great fortune, he had never gone "broke". The company owed nothing and had plenty of money to go ahead with its work. This firm was solicited for its bond. The bonding agent repeatedly urged the President to close the transaction with him, and finally was told to write the bond. When the agent sent the bond to his main office, they refused to write it, and at the start of its business career this new company was placed in a bad light with the State, which was about to award it the contract. Fortunately, another bonding company wrote the bond which amounted to $3 000. The firm has been paid for its $3 000, the contract is completed, and, more wonderful than all, the contractor made some money on his work.

HIGHWAY FINANCING AND BONDING

By Herbert S. Sisson,* Esq.

The subject of automobile taxation has a vital bearing on the highway situation in the State of New York. The speaker believes it is estimated that in 1922 the total revenue from automobile taxation in this State will be more than $10 000 000, which would average about $12 per vehicle. The law provides that a motor vehicle in the State of New York may not be taxed as personal property, which means that the value of the automobile is not taken into consideration as personal property and taxed as such. The only tax on an automobile, or that an automobile owner pays on the money invested in the machine, is his license tax. This, the speaker believes, is not the case in some States. He does not believe that the automobilists of the State of New York should pay for the entire cost of construction, reconstruction, maintenance, and administration of the Highway Department, but he does believe that they should pay more money than they are at present toward the reconstruction and maintenance of highways. Approximately, $9 000 000 or $10 000 000 each year are being spent for the maintenance and reconstruction of the highway system. That will probably increase for a few years at least as the mileage of completed highways increases.

It is hoped that the type of highways now being built and which have been built for the last few years may not require the large sums per mile that are now being expended on some of the earlier built highways. That is a hope, however. The speaker does not believe that any one can foretell what the traffic conditions may be 10 or 15 years from now. Perhaps the highways that are built to-day will be as inadequate 10 or 15 years from now, as the highways built 10 or 15 years ago are to-day.

In discussing this subject, a Highway Commissioner of one of the counties bordering on the City of London, England, described the type of highways it was necessary to build in that locality, and particularly called attention to the kind of traffic that those highways were called on to bear. He spoke of the motor lorries, propelled by steam, which have steel tires and carry such immense loads. If that type of motor vehicle is ever introduced in the United States, and used to any extent, the speaker has grave fears as to what will happen to even the best of highways that have been built or that are being built.

The speaker believes that no one in the State of New York seriously questions the advisability of a reasonable increase in the motor vehicle tax. No one intends or expects that the motor vehicles in this State are to pay the entire cost of construction and reconstruction of highways, but it is felt that the motor vehicles should pay a somewhat larger tax than they are now paying for the purpose of highway maintenance.

The speaker wishes to discuss briefly the bonding of highway contractors in the State of New York. Mr. Greene has called attention to the bill that was passed in 1920, which allowed a contractor to dispense with bond by agree-

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ing that the Highway Department could retain 20% of the estimates, as the work progressed. This works, in the speaker's opinion, as follows: When a contract is to be awarded to a contractor who is low bidder and who elects to dispense with bond, the Highway Commissioner must be the judge of whether he is financially able to carry on that contract to completion.

In case he is a new bidder and has never had a State contract, there is no way of determining whether he is financially able, and has the business ability to carry on that contract and to complete it. The speaker believes this is an unfair burden to place on the Highway Commissioner.

Furthermore, a contractor who has never had a highway contract in this State, or perhaps, in any other State, immediately finds that he is not obliged to give a surety company's bond, and immediately concludes that this is a way of saving or increasing his profits to the extent of the premium that he would pay to the surety company for a bond. That is so, to a certain extent, on the basis mentioned by Mr. Greene, except that the borrowing ability of the contractor on the retained percentage basis is much impaired. With the contractors who have dispensed with bond, there comes a time usually when it is difficult for them to borrow money for that reason, and many of them state that, unless they can have a semi-monthly estimate, which is contrary to the rules, they will have to stop work. That has happened in one or two cases, and the work was stopped until the monthly payment was made. The speaker believes that, in those cases, if the contractors had been bonded, they probably would have been able to obtain money either from banks or other sources.

Under the Hewitt Act, which was passed in 1921, the State of New York will have a highway system comprised of 11 260 miles. There are either completed, or under contract, all except about 2 200 miles of this road. The question of financing the construction of these 2 200 miles has been seriously discussed, not only by the Highway Department and the present State officers, but by the past administration, and the speaker thinks it is agreed by every one who has considered the matter seriously, that it will be unnecessary to issue further highway bonds for construction purposes.

The long-time bond has become obsolete in the State, by law; and, with its great resources, the State of New York will be able to finance the building and the completion of this great system without a further issue of bonds. The people of the State of New York, and of other States, have come to the conclusion that not only in their own private business, but in public business, the policy of "pay as you go" should be adopted wherever it is practicable.

A large sum of money will be required to build this highway system, say, $75 000 000. Of the second $50 000 000 bond issue, there is about $6 000 000 unexpended, of which from $4 000 000 to $4 500 000 will be used on construction in 1922. The remainder of this year's construction will be financed either from what is known as Federal Aid, that is, the amount contributed or appropriated by Congress for the aid of the States, of which this State gets about 5% of the total appropriation, which must be matched by an equal amount appropriated by the State Legislature. The amount available in New York in 1922, together with the bond issue, and probably a small direct

appropriation, will finance the construction of the coming season. The continuation of aid from the Federal Government and a direct appropriation by the State to match it, and another direct appropriation of some millions of dollars, will finance the program for 1923. The Hewitt Act provided that the entire system of 11 260 miles should be completed within 6 years from the passage of the bill. The rate of progress made during the season of 1921, and the expected progress during the season of 1922, will be on that average basis.

All these matters of finance, from the standpoint of the Highway Department, are the cause of much work and study. The distribution of the money and the work in the several counties of the State is fixed by law. First, the last $50 000 000 issue was divided among the counties and, for that reason, the $6 000 000 of that issue can never be entirely used. There will be balances of from $1 000 to $10 000 in the different counties, which will not be enough to finance any particular piece of work in a county. For that reason, about $1 500 000 or $2 000 000 can never be used, and for which, therefore, the bonds can never be issued.

It is not known whether future Congresses are to continue what is known as Federal Aid. If they do not, other means of financing the completion of this system will have to be devised by the authorities of the State of New York.

THE MOTOR TRUCK AS AN ASSET TO RAILROAD OPERATION

By R. S. Parsons,* M. Am. Soc. C. E.

During 1922, more improved highways will be constructed in the United States than ever before in a similar time. Federal, State, county, and city appropriations, aggregating more than $1 000 000 000 are being set aside for this purpose. It is the passenger vehicle that makes road construction a question of popular interest and this enormous undertaking possible, but it is the motor truck that sets the standard of road construction and makes the expenditure of economic interest. Railroads are concerned in this work, as they are taxpayers to the extent of more than $200 000 000 yearly, a large part of which goes into the highway budget.

The motor truck has demonstrated its ability, as a mechanical device, to move freight promptly and efficiently. Consequently, railroad managements must accept the inevitable and meet the motor truck as an ally rather than as an enemy. Inasmuch as a benign public has seen fit to provide the right of way, roadbed, and track for this means of transportation, it is the duty of the far-seeing railroad manager to welcome it as an aid and not condemn it as a competitor.

Some of the conditions under which motor-truck operation can be substituted for present-day railroad operation are:

1.—Short branch-line operation.
2.—Trap-car service.
3.—Suburban distribution.
4.—Utilization of outlying yards in lieu of yards in congested districts.
5.—Terminal distribution.

Short Branch-Line Operation.—Branch lines of light traffic are unprofitable. Motor trucks, on improved roads, can be substituted for both passenger and freight service and the railroad lines can be abandoned. If improved roads do not exist, the right of way and roadbed of the branch line can be turned over to the proper authorities for the construction of a highway. An instance of the use of this arrangement may be found in the utilization of a part of the abandoned roadbed of the Delaware, Lackawanna and Western Railroad by the Pennsylvania Highway Commission for the State Road between Binghamton, N. Y., and Scranton, Pa.

Trap-Car Service.—In industrial centers, many industries produce freight for diversified points. This freight must be assembled at some central station and consolidated for various destinations. The movement of the cars of freight for miscellaneous points, between the industry and transfer platform, is known as trap-car service. It is usually performed through congested yards, and in order to render proper service must be performed on a specified schedule This renders the service expensive, and the cars used in this manner take up unnecessary room in the yards to the detriment of through service. This trap-car service can be performed by motor trucks much more cheaply than

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by the use of cars. The result is an expedited movement with real economy, a saving of equipment, and relief of congested yards to the advantage of carload outbound freight and inbound raw materials.

Suburban Distribution.—Every large city becomes surrounded by a suburban area of more or less intensive development. The use of the expensive terminals of the railroads for local distribution throughout these areas results in an economic loss and requires the development of much larger terminals than would otherwise be necessary if such distribution were eliminated. Freight for local distribution cannot be refused, however, as such action would affect the solicitation of profitable long-haul business. Terminal costs more than absorb the entire freight rate on local distribution of this character. Motor-truck distribution through suburban areas, within a radius of from 30 to 50 miles (depending on the condition of the roads), is more satisfactory and economical than similar distribution through the medium of city freight-houses and way freights. This method of distribution in the New York suburban area should be cultivated by all railroads.

Utilization of Outlying Yards in Lieu of Yards in Congested Districts.—In rare instances, railroad managements have been far-sighted enough to anticipate the growth of cities and to prepare adequate yard room to accommodate the increase in business. Where this has not been done, it results in later years in the necessity of purchasing areas at great additional cost and under disadvantageous circumstances. The growth of a city compels much additional expense on account of the elimination of grade crossings and diversion of highways; also, municipal ordinances against smoke, noise, and the blocking of crossings, make the operation of internal yards extremely expensive. The elimination of such yards; the selling of the valuable property released thereby; the use of funds so acquired in the construction of more properly designed yards outside the city limits; together with the institution of motor-truck service to interior stations, would undoubtedly prove economical in many instances.

Authorities agree that the proper measure of cost of operating motor trucks is the cost per day rather than the cost per mile. As the daily mileage increases, the cost per mile will decrease. There are so many items of this expense, that are not absolutely dependent on the mileage made, that an additional mile—or even two miles—between outlying yards and city freight-houses will not increase the cost of trucking service proportionately.

Terminal Distribution.—The growing opposition to grade crossings and the expense of eliminating them; the increase in municipal and State taxes; and the enormous cost of real estate, render the building of industrial tracks in the heart of great business centers impossible. Here, the motor truck becomes a necessary aid in combating undue terminal charges.

In St. Louis, Mo., with its population of nearly 800 000 people, practically all the railroads terminate their freight activities at East St. Louis, and all carload and less than carload freight, where there is no direct connection with an individual industry, is contracted by a trucking corporation, which provides inland stations and delivers carload freight and truck freight from East St. Louis across the bridge to St. Louis.

The tractor and trailer system which is used at this point, delivers carload and less than carload freight between the railroad and the consignee on inbound and outbound freight. This system takes up the lost motion on the motor unit, three trailers being provided for each tractor. The haul at this point averages about 2 miles, and the tractor with its three trailers can handle about 45 tons of freight per day, at a cost of about 70 cents per ton.

As a demonstration of the economy of this method, it might be cited that only a short time ago the Pennsylvania Railroad Company, with an all-rail line connection in St. Louis, diverted its freight to this tractor-trailer method of distribution.

In Cincinnati, Ohio, the demountable body, or container, is used, the operation being confined to the transferring of less than carload freight from one railroad to another. The demountable body has the same advantage as the tractor-trailer in allowing the motor to continue in operation while the loading and unloading takes place. Hoists, at loading and unloading points, quickly remove the container from the truck chassis. The average haul is about 1½ miles and the cost is about 51 cents per ton.

In New York City, a radical departure is being made at the present time in terminal distribution of inbound freight and the collection of outbound freight. Under a contract with the United States Trucking Corporation, the Erie Railroad Company delivers its cars to the Trucking Company in its yards on the New Jersey shore. The freight is loaded on trucks and ferried to New York City, utilizing the ferries, in so far as possible, at times other than during peak loading. A common rate is paid on all freight trucked to an imaginary point in West Street. From this imaginary point, freight is distributed: First, to piers for steamship loading; second, to inland freight stations; and third, for store door delivery. Different additional rates are allowed for these three methods of distribution. In the case of steamship delivery, freight is unloaded on the docks the same as any city freight that may be so handled.

In the second instance, inland freight-houses have been developed by utilizing the ground floors of warehouses, owned by The Independent Warehouse Company, at Beach, Greenwich, Leroy, and Montgomery Streets. Through an arrangement with the Warehouse Company, freight may be floored for immediate delivery to the consignee or may be elevated into warehouses at the consignee's request for subsequent delivery. These inland stations are in every respect railroad stations.

The third method of delivery is worked out between the Trucking Company and the consignee, the latter preferring, in many instances, to receive his freight from the Trucking Company rather than have it go through the inland stations. This reduces the cost of handling freight both to the Trucking Company and consignee and expedites delivery. True economy, however, cannot be reached in this work until specially constructed trucks of either the "container" or "trailer" type have been placed in the service.

With the completion of the Vehicular Tunnel and the elimination of the ferry service with its resultant delays, this method of delivering freight to New York City will meet with increasing favor. It is a well-advertised fact that it costs as much to move a carload of freight from the railroad yards in

New Jersey and deliver it in New York, as it does to move the same car of freight from the same yards to Buffalo, N. Y., or Pittsburgh, Pa. Any method, therefore, which reduces this large expense (which is principally in the maintenance, operation, and repair of lighters, tugs, and docks), will be welcomed.

There has been a noticeable shortage of steamship pier space on the North River for some years, due to the fact that 30% of all the piers are being used by New Jersey railroads for the purpose of delivering freight to New York City. Through the trucking method of delivery, a large proportion of these piers will be released and can again be used for steamship purposes. It is a pity that this proposition had not been worked out before the steamship piers on Staten Island were built, as the burdens which will be imposed on the railroads for a long time to come through lightering to this distant point, will be a marked reflection against the economical operation of the port.

The New York-New Jersey Port and Harbor Development Commission contemplates the construction of vast assembling yards on the New Jersey meadows, where cars from all railroads will be assembled; extensive transfer stations, where freight will be taken from cars and loaded in containers for transfer to tunnels; an extensive tunnel system, where small cars automatically and electrically operated will transport this freight to points underground in New York City for further handling and distribution; and warehouses and freight stations in New York City to facilitate this distribution.

In this construction more than $200 000 000 must be spent and when it is in operation, any freight so handled, after it has reached the city, must ultimately depend on the motor truck to reach its destination, the only exception being where the receiver of freight is fortunate enough to be located adjacent to one of the terminal stations.

With the Vehicular Tunnel from Canal Street, New York City to Jersey City in operation, freight may be loaded directly from cars in New Jersey to motor trucks and without further re-handling be distributed directly where required by the receiver.

Except for the North River, the Port Development Plan would not meet with a moment's consideration. Let us, therefore, undertake to eliminate the North River by the construction of a series of tunnels, at a cost much less than the Port Plan.

First.—Build a tunnel from Lower New York to a point adjacent to the existing freight yards of the Jersey Central, Lehigh Valley, and Pennsylvania Railroads.

Second.—The new Vehicular Tunnel, now under construction, between Canal Street and a point adjacent to the yards of the Erie and Lackawanna Railroads, will take care of freight unloaded at these points.

Third.—Build a tunnel from a point near 50th Street to a point adjacent to the West Shore Tunnel, for the handling of the business of the West Shore.

Fourth.—Build a tunnel from 125th Street to a point adjacent to the freight yards in the vicinity of Fort Lee Ferry.

Fifth.—Build a belt line along the Hackensack River to the New Jersey Meadows, connecting with all the railroads mentioned. At the present time,

tracks are available for about half this distance. Over this belt line, freight consigned to New York City could be moved to the most adaptable yard, unloaded at existing team tracks, and trucked directly to point of destination, eliminating any north and south trucking in New York City.

This plan, as outlined, would cost much less than the plan under consideration and would give relief to the entire city instead of to a small part of it, and, in addition, would expedite the movement of pleasure vehicles in and out of the city. Trucking of this nature would not add to the congestion of New York City streets as the same freight is now trucked between warehouses, ferries, and the pier railroad stations. The advantage to the railroads lies in the elimination of the tremendous cost of lighterage, the two additional handlings now necessary in loading and unloading the lighterage, and the incidental saving of loss and damage account, a large proportion of which accrues at transfer points. The latter item for all railroads during 1921 amounted to more than $100 000 000.

MOTOR VEHICLE CONTROL IN MARYLAND

By G. Wythe Munford,* Esq.

Motor vehicle control, as it has been evolved in Maryland, to keep pace with the development and steadily increasing use of the automobile, is now grounded on four essential requirements. Although continued expansion of motor vehicle traffic may require enlargement of official regulatory power, experience leads to the belief that these four requirements are basic and essentially permanent.

Proper and effective regulation and control requires that close check at all times be kept on the individual car once it has been identified and recorded. This record of the car as it goes into the "used car" class must be permanent. Ready identification and a maximum of ease in tracing to its owner any car seen on the highways must be provided. Before being turned loose on the highways the driver, for the protection of himself and to safeguard others, must undergo some mental test and give practical demonstration of his fitness and ability. Once permitted to operate, the driver at all times must be easy of identification and location. These requirements constitute Maryland's four basic principles of motor vehicle control. These ends are accomplished in the following manner: First, there is a Title Law; second, the ordinary system of yearly registration and issuance of a numbered metal tag is used; third, each chauffeur or operator must submit to mental examination and give practical demonstration of his or her ability to operate a car; and, fourth, every driver is assigned a permit to drive and given a number.

The Title Law, although it is mentioned first, constitutes the latest addition to the system. In addition to its other advantages, it has done much to check automobile theft. It has made the "auto Jack's" vocation more hazardous and correspondingly less profitable. Under it, no Maryland automobile, motor-cycle, or side-car can be sold or bought in the State unless there has been an exchange and a recording of a title deed between the vendor and vendee. The transaction of sale is conducted in manner somewhat similar and is, indeed, supervised more closely and protected more completely than an ordinary sale of real estate. Not only must the deed or title of ownership contain a minute description of the external characteristics of the car; it must also identify the engine or motor by its original factory number.

Such a statute, of course, would be useless if it did not contain heavy penalties. Provision for a fine of as much as $1 000, or a jail sentence up to 7 years, or both fine and imprisonment, for the mere having possession of a motor vehicle from which "the trade mark, distinguishing or identifying number, serial number, or marks has been changed or is changed, covered, removed, defaced, destroyed, obliterated or altered", is indication of the severity of the penalties.

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Forgery in application, misstatement in application, or any alteration of a title of ownership once issued, carries penalties of from $1 000 to $5 000 fine, or jail sentence up to 10 years, or both fine and imprisonment. Equally deterrent penalties are provided for having in one's possession a car or cycle for which title has not been issued.

All titles of ownership are issued and a record of them kept in the office of the Commissioner of Motor Vehicles. The same requirement governs all transfers or assignments of title. Through this system every motor vehicle in the State can be traced back to its original owner. Its present owner's name and address is at all times a matter of public record.

If a car is stolen and its motor number is changed or obliterated before it is recovered, or, in cases of cars which had been sold prior to the passage of the Title Law, with bad numbers, the Commissioner assigns that engine a new number. This substituted numeral, with the identifying initials "A. C."—for Automobile Commissioner—added, is indented on the motor. By it, the car is known and identified in all subsequent sales.

The title records give the authorities at all times a complete and a permanent description and record of present ownership and approximate location of every motor vehicle in the State. This record is independent of the system of yearly registration of all cars and assignment to each of two numbered metal tags which must be displayed at all times—one in front and one at the rear. When these tags are issued yearly, they are assigned to a specified car. They cannot be used on any other. They will not be issued except in the name of the persons whom the title records show to be the owner of that particular car. These registration numbers are issued for the calendar year, or any quarterly proportionate part thereof.

The system of yearly registration and identification tag issuance goes further, however, than the simple assignment and issuance of numbered display tags for road identification. A card describing the name or make, the color and type of body, the color and type of wheels, and containing also the factory-given engine number, as well as the tag numerals assigned it for that year, must be carried at all times with the car. This card is issued by the Commissioner. On it is also stamped the ownership or title reference in the files. This system, for violation of which severe penalties are provided, gives at all times speedy identification and tracing of a car while it is on the highways.

Of course, the inanimate car or motor-cycle is thus at all times so marked as to be easy of identification and location after an accident or traffic violation. It is, however, after all, the driver who is wanted in such cases.

The Maryland law endeavors, first, to keep all persons from behind the steering wheel and away from the gear control of a motor vehicle until he or she has demonstrated ability to operate a car, with the minimum of danger to others, and has a working knowledge of traffic regulations and the rules of the road. This is accomplished through the issuance first of a beginner's license or instruction permit. Under this permit, the new driver is forbidden to operate a car unless a fully licensed and experienced operator or chauffeur is present. This permit is good for only thirty days, but may be renewed, and often is extended if the learner has not become proficient in that time.

After the period of instruction and actual experience in operating a car, always under the eye of an experienced driver, the applicant reports for an examination. Specific dates are advertised in the counties for these tests, which take place usually at the county seat. Demonstrators or examiners are on duty for this purpose at all times in the Headquarters Office in Baltimore. First, there is an oral test to determine what the applicant has learned of the working parts of his vehicle, and what his conception of his own and the other person's rights are on the highways. Passing this successfully, the applicant gives a demonstration. With a representative of the Commissioner on the seat with him, he is compelled to start and stop the car, take it through fairly heavy traffic, back it, stop after stalling on a hill, etc. If the applicant cannot meet the standard in these tests there must be a further period of operation on an instruction permit, another test, and another demonstration.

The fact that many applicants are often sent back for instruction two and three times indicates the importance of this examination and the requirement for a standard of qualification. Some have never been able to qualify. The weight that is given to proper preparation and qualification of drivers is indicated through the fact that, in cases of sub-normal mentality or a suspicion of such a condition, the Commissioner can require the applicant to go before a board of physicians for examination. Sixteen years is the minimum age limit for drivers.

Having set a standard that "would-be" drivers of motor vehicles must meet before being turned loose on the highways, Maryland lists and indexes every chauffeur and operator in the State, and issues him a numbered card on which is his name, address, and a description of his physical characteristics. This card must always be carried and must be shown on demand, following any violation or accident. Demand that the operator's card be shown under such circumstances can be made by any one, not necessarily an officer of the law, and refusal to do so entails severe penalties.

As a further protection, these permits are subject to suspension for various periods or to revocation by the Commissioner. All such suspensions and revocations are handled by the Commissioner. The magistrates trying cases of violation are limited to imposition of fines or jail sentences. A magistrate's acquittal on a charge, however, does not estop the Commissioner from independent action. He is the final judge of the right of a driver to retain his permit. He conducts independent investigations of violations and often revokes or suspends the license of an operator who may have been exonerated by a trial magistrate.

As a check on the record of drivers, all magistrates are required to make detailed reports to the Commissioner on all cases tried by them. From these reports, with the disposition of the case, a complete file of difficulties or violations of every driver is centrally assembled. In this way, each driver's record of conduct, and a determination of second or subsequent offenses, is made constantly available.

Suspension or revocation of a driver's permit denies him or her the right to operate within the State. Violation of this section is penalized by a fine of

$100 to $1 000, or a jail term of from 30 days to a year, or both fine and imprisonment.

In each of these stages of regulation, except in the case of the examination and demonstration by the driver, a fee is collected. The drivers are grouped into three classes: Chauffeur, including those who drive for hire; operator, who drives for pleasure or operates his own car; and motor-cycle operator. These pay, respectively, $3, $2, and $1. Issuance of each title or transfer of title entails a cost of $1, plus the notarial fee. There are 331 565 drivers of all classes paying $787 695 annually, the increase in 1921 being more than 25 per cent.

Charges for registration and issuance of marker tags are based on the rated horse-power for pneumatic tire vehicles and on a straight-tonnage basis for solid-tired cars. Any vehicle with two non-pneumatic tires is rated as solid-tired. The pneumatic-tire rate is 60 cents per h. p. for pleasure cars. Hiring cars or taxicabs pay twice this rate.

The system of registration regulation and basis of fees has one exception. This exception constitutes the only point in which there is a division of jurisdiction between the Motor Vehicle Commissioner and any other governmental department of the State. It has to do with franchise rights and the regulation and costs for operating passenger and freight bus lines over established routes on arranged schedules of service. In this case, the Motor Vehicle Commissioner and the Public Service Commission, controlling all public utilities in the State, share administrative functions. The speaker will touch on this phase of motor control, as Maryland now handles it, more fully subsequently.

From all the fees collected, with the exception of those derived from the Titling Department, is created, after the administrative expenses of the Motor Vehicle Commissioner's office are met, a fund devoted to the maintenance of the State's highways. None of it goes to construction; it is merely applied to putting back into the road's surface what the automobile takes out of it in wear and tear.

Some idea of the growth and the increased requirements for the regulation of motor vehicles in Maryland may be gleaned, however, from a comparison of the gross income from motor vehicle registration when Maryland first attacked the problem in 1911. Determining to lay down a system of modern hard-surfaced highways, on which $26 000 000 has since been spent for construction, the State turned to automobile license fees to take care of these roads after they were built. In that year, or in the succeeding fiscal year, the gross receipts were $56 204. Gross receipts in 1920-21, as shown by the annual report, amount to $2 451 166.08, an increase of more than 400% in 10 years. The administrative problems are multiplying more rapidly than the income. This is best shown by the fact that, in the calendar year 1921, 18 000 more pleasure cars alone were registered than in the preceding twelve months.

In view of this discussion the attitude of the State of Maryland relative to both motor vehicle control and highway construction and upkeep, may be of interest. Although recognizing the inevitability of continued expansion

in the field of truck transportation, Maryland is now discouraging heavy unit tonnage transported on solid-tired vehicles. With mounting tonnage on solid tires, the fees are being sharply boosted. Beginning with the 1-ton truck, the State collects $20 per ton, up to a 3-ton vehicle. Then the rate goes up: $100 for 4-ton trucks, $150 for 5-ton, $300 for 6-ton, and $500 for 7-ton vehicles. However, although the State is still licensing 6-ton and 7-ton trucks which were owned and registered in Maryland prior to 1918, it now has a 5-ton maximum for all other cars of this type, and limits the gross weight to 20 000 lb. Even with these restrictions, 11 597 solid-tire trucks were licensed in 1921, the fees totaling $371 000.

In the realm of motor vehicle control of all trucks, the State has special provisions. To enforce the gross weight law, roadside scales of the ordinary type for vehicle weighing have been installed at strategic points on through lines of traffic. To these scale boxes, the motor-cycle police have keys, and any truck under suspicion of being over weight must submit to weight on demand. Fines of as much as $100 for each over-weight truck are provided.

The State also has a maximum width law for trucks, which limits the width to 90 in. The speed limit for solid-tired trucks is decreased as the tonnage increases. The carrying by all trucks of a mirror adjusted so as to give the driver a view of the road to his rear, is also compulsory.

The truck-regulation provisions just cited cover both the free-lance truck, which will, as they so often advertise, haul anything, anywhere, anytime, and the regularly scheduled freight-hauling motor.

In the latter type of vehicle, however, as well as in the case of the passenger bus covering an agreed and fixed route on a regular schedule as a common carrier, the administrative jurisdiction is shared with the Public Service Commission.

The Service Commission controls the franchise, compels the observance of schedule, fixes the freight and passenger tariffs, and hears and adjudges controversial questions arising between the operating company and its patrons.

The Commissioner of Motor Vehicles assesses and collects the license fees for registration, issues and enforces observance of the provisions of chauffeurs' permits, and compels observance of the traffic regulations. He also issues the registration identification tags, which are distinctive for this type of vehicle, and administers the Titling Law.

The passenger bus in this service is licensed on the basis of one-twentieth of 1 cent per passenger-mile for the entire scheduled route and for the full capacity of the vehicle, for buses weighing less than 3 000 lb. Buses, weighing between 3 000 and 7 000 lb. pay one-fifteenth of 1 cent per passenger-mile, and those weighing more than 7 000 lb. are assessed one-sixth of 1 cent per passenger-mile.

For freight-carrying buses, the fees which are collected on a ton-mile basis, are, as follows: One-fifth of 1 cent for a 3-ton truck or less; two-fifths of 1 cent for trucks weighing between 3 and 6 tons; and three-fifths of 1 cent for all trucks of more than 6-ton gross capacity.

The joint regulation of such traffic is not limited to rural service. It applies also to the Charles Street Bus Line of the United Railways and

Electric Company, in Baltimore, and to all jitney lines operating in that city and the smaller towns of the State.

From 260 buses of various types so licensed the fees for 1921 totaled $44 734.18. That this business is also growing is indicated by the fact that this sum was exceeded by nearly $1 400 in the first license rush at the beginning of January, 1922. With the opening of the many rural resorts, to which such lines are in increasing numbers devoting their attention, the spring and summer will see an even further increase.

This source of expected further increase does not indicate any limitation of chartered bus and truck lines to summer-time operation for rural resort service. In Maryland, as elsewhere, the passenger, baggage, express, and freight motor traffic is well out of its swaddling clothes, although as a rural highway transportation development it is still treated as somewhat of an infant industry.

Because of this, and because it is looked on as an important and effective instrument for rural community development, the State is protecting all properly financed and efficiently managed lines; it protects their lines and routes and discourages unstable and irresponsible competition.

In large agricultural areas, devoid of either rail or water transportation, motor-bus lines are now furnishing for the first time in the history of these localities satisfactory and convenient connections with the outside world.

As distinguished from scheduled and routed bus lines, the 4 161 taxicabs and hiring cars not operating over any fixed routes paid in 1921 in fees to the State $110 663.65. Pleasure cars, of which there were licensed 120 231, added $1 468 969.04 to the State's income. Income from State dealers, dealers in transit, motor-cycles, and other sources, might also be discussed.

Over and above principles and requirements in the theory of motor vehicle control, because these are useless and practically unenforcible without them, is the body of uniformed, trained, and motor-cycle mounted police. The psychological effect of such a force in restraining the potential "speed king" and reckless driver can hardly be estimated.

In addition to the safety they bring to rural highways, their contribution to motor vehicle control results in the collection of fines sufficient to pay their daily running expenses.

DISCUSSION

T. Hugh Boorman,* Esq.—Inspection is one of the most important factors in highway work. As has been stated, and as every one realizes, the greatest work in the United States to-day is the building of roads.

In reviewing more than forty-nine years of service, one realizes how methods of transportation have changed, and to-day it is important to have good roads and streets. The education of the inspector, as has been remarked, should start early in life; young engineers on being graduated must do something practical, and the speaker does not know why it is not just as well to start on inspection as any other line of their necessary practical experience.

Good inspection is important, and it will be, and undoubtedly is, advisable that an endeavor should be made to train young engineers to feel that a road inspector is a man who is going to be of immense service to the country.

Samuel Whinery,† M. Am. Soc. C. E.—All that has been said about the importance of good inspectors and the difficulty of obtaining them is true. On almost any public work, whether it is the building of a road, a railroad, or a bridge, there is no more important position on the whole job than that of inspector, provided he is of the right kind. The practice in the United States has largely been to select young technical graduates for the position. As a rule, they are an excellent class of men. They have had a good technical education, and it is natural to assume that they are well prepared to be competent inspectors on any kind of engineering work. So they are, in some respects, but not, the speaker thinks, in those of most importance. His own experience has been, particularly in railroad, bridge, and masonry work, that the most valuable inspector is one who is an expert in the particular kind of work he is to inspect, although he may have but little technical knowledge.

For instance, on masonry, the best inspector the speaker ever had, was a man who never attended college, who began early in life as a stone mason and followed his trade until manhood. There was nothing about any kind of masonry that he did not know. In addition, he had good judgment and the faculty of doing things in a pleasant way. Contractors soon came to appreciate that, although fair and reasonable, he knew his business thoroughly, and that his instructions were the law. The speaker remembers another man, later general inspector of masonry on one of the largest railroads in this country, who advanced in the same way. He was a practical mason, trained in every branch of his work.

The trouble with young engineering graduates is that they have had no opportunity to learn from experience the matters of practical detail that, after all, are likely to make up the bulk of an inspector's duties. They understand the requirements of the specifications, which cannot always be literally complied with, but do not know when these literal requirements, in matters of detail, may or should be varied from in the interest of good work. The speaker has had similar experience in street and road work. The man who will be most useful as an inspector is one who knows the entire process of

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† Cons. Engr., New York City.

road building. Without disparaging in any way the merits, and they are many, of the college graduate, the speaker would say that it seems unjust and unfair to place him in a position for the duties of which education has only partly fitted him and in which he has had no practical experience.

The value of a technical education as a qualification for the position of inspector on public work may well be questioned. Aside from the mental discipline that a course in college develops, or should develop, it does little to qualify a young man for the duties of inspector. Such duties are almost wholly of a practical nature, a knowledge of which can be acquired only by experience, observation, or special instruction, regardless of technical training.

The speaker has had some experience with inspectors in cities and on country road work. The chief trouble, of course, results from the appointment and retention of incompetent men for political or personal reasons. Even in cities where Civil Service examinations are required, one will find, as inspectors on the streets, men who are incompetent and whom one would not trust in any responsible position. They are ignorant of the duties required; they serve the interests of political bosses rather than those of the city; and, too often, they accept, if they do not demand, gratuities from the contractor.

About ten or twelve years ago, the speaker was asked to report on paving conditions in one of the large cities of the United States; and, among other things, he looked into this matter of inspectors. He thinks he is perfectly justified in saying, not from suspicion, but from proof presented, that three out of five inspectors on the streets received regularly weekly pay from the contractors. In another city, where he was somewhat in responsible charge, and where he found incompetent inspectors and was able to secure their dismissal, he would find them, a few days later, as inspectors on a street in some other part of the city. Incompetent and dishonest inspectors give the unscrupulous contractor an opportunity to "scamp" his work and then attempt to evade responsibility by claiming that it was done under the supervision and with the approval of an agent of the municipality.

As the result of his experience and observation, the speaker believes that the best inspectors, for nearly all kinds of public work, come from the ranks of intelligent and honorable men, who, as craftsmen, have acquired a practical knowledge of the work.

JOHN C. TRAUTWINE, JR.,* M. AM. SOC. C. E.—As the papers on the subject of highway transportation have dealt largely with the economic or sociological side, the speaker will discuss briefly the sociological aspect of privately conducted water and rail transportation, as contrasted with publicly conducted highway transportation.

The people, to-day, are in a situation, the like of which the world has never before seen. What the speaker will call the "ancient" system of individualism and the "modern" system of socialization are still traveling side by side, like a muddy and a clear current in the same stream.

Under the "modern" system, the community gives to each one regardless of merit, all the highway facilities there are, and one can get no more of such

* Philadelphia, Pa.

facilities by over-reaching his neighbors. Thus, the tendency to over-reach, which is the soul of private business, is becoming atrophied for want of use. Even in the purely socialistic furnishing of public highways for the free use of all, an operation in which profit is out of the question, the alleged necessity has not been outgrown, of employing the profit-seeking contractor. The "ancient" system is still followed which places its premium on self-seeking, thus making inevitably for immorality. It is only in so far as the "modern" system is thus smirched by its contact with the "ancient" system, that "graft" exists. The "modern" system is pure. Governments approach respectability in proportion as they assume new and greater public service functions.

The inevitable transition, from the "ancient" to the "modern" system, is being brought about by the irresistible push of universal demand, the selfish demand for better service than the "ancient" system can give. It is significant that this relatively "modern" discussion on highway transportation has dealt with the more "modern" highway system, under which a public servant, caught playing the "business man" and making a profit out of his position, would be disgraced forever.

Note these two illustrations, one taken from the "ancient" the other from the "modern" system, the two showing the contrast between the moral standards of the two systems:

(1).—During this discussion, it has been stated that, under the "ancient" system, it is considered not at all irregular for a railroad line, in competition with a water line, to reduce its rates to the losing point, and to keep them there until it has killed off its water competitor, whereupon it proceeds to charge "all the traffic will bear".

(2).—Contrast this with the attitude of the Pennsylvania Highway Administration, as outlined by Mr. Hilts, and its attitude toward the contractor. Far from taking advantage of the contractor's ignorance of conditions, it strives to keep him out of difficulty, knowing that, in difficulty, he cannot serve efficiently.

Eugene W. Stern,* M. Am. Soc. C. E.—In connection with the question of highway transportation it may be in order to speak of the imperative necessity of having only highly capable men in responsible charge of highways. Too much cannot be said to the public for their own good in this regard. Men of only the highest qualifications should be employed, and such men cannot be retained unless they are paid adequate salaries and are freed from political subserviency.

For years, the speaker has noticed that the lack of intelligent vision and foresight on the part of chiefs of various highway bureaus has been responsible for the construction of highways utterly inadequate for the traffic and hence extremely expensive to the taxpayer. He has seen roads of an inferior type built to carry heavy and ever-increasing traffic, which should have been built to a standard almost equal to that of a high type of city pavement. On account of the vast sums of money now being expended in highway construction, and which will increase as years go on, it is in the power of the men chosen to

* Cons. Engr., New York City.

head highway departments, by poor judgment, by improper selection of types of roads, by incompetent and negligent supervision, to inflict on present and future generations, enormous annual excess charges for upkeep, over and above what proper construction, etc., would have warranted.

The failure to make the right decision in the selection of types of roads by those who had the power to do so, is causing losses of millions of dollars a year to the taxpayers of the State of New York. The speaker does not blame the present, nor the past administrations, nor any particular administration, for this state of affairs. The motor vehicle had to be accommodated, and provision was not properly made for it in the writing of the specification for many roads, on which enormous sums for annual maintenance charges are now being paid.

Lack of vision is often shown by highway commissioners in neglecting to improve the alignment, gradients, etc., of old roads. A 16-ft. roadway is known to be not wide enough for safety on an important highway. Why continue to build them? Why build roadways 16 ft. wide on curves, when they should be at least 26 or 30 ft. wide, to safeguard life? An old highway built for horse traffic a century ago, for instance, is being improved with a hard surface for motor vehicle traffic. Why is the location of this road not changed, the alignment corrected, the curves made easier, the gradients improved? Lack of vision and intelligence on the part of those who are chosen to safeguard the interests of the people in road construction, are responsible for these deficiencies.

The speaker will mention a case in point: The main thoroughfare called the Albany Post Road, which leads north from New York City along the east bank of the Hudson River through Tarrytown and Peekskill to Albany, has many errors in location and construction. In this regard, it is the worst that the speaker has ever seen on any stretch of important road. He has been over many miles of roads in France and has never seen a poorer piece of location, even on the unimportant Communal roads (comparable to American town roads). Many defects of alignment, curvature, and grades could have been eliminated when the road was being changed for motor traffic. There are many places dangerous to life. On a main thoroughfare on which the traffic is enormous, there should never be such names as "Dead Man's Curve", etc.

If the speaker has made some impression, it is hoped that it may have the effect of bringing to the minds of those responsible for the selection of men to head highway bureaus, the importance of having only the highest and most competent type of men; that they must be allowed practically a free hand and not be interfered with by politicians; and that, unless this is done, the waste of money due to deficiencies in construction and maintenance of highways, in this coming generation, will run into enormous sums annually.

HENRY GOLDMARK,* M. AM. SOC. C. E.—The speaker will discuss the condition of highway bridges as distinct from the highways themselves. He is led to do so more especially by a personal experience connected with one of the main highways in the Adirondacks over which he has had occasion to travel for

* Cons. Engr., New York City.

more than thirty years. As the amount of traffic and the weight of the vehicles have increased, the road has been gradually improved so that now it has an excellent macadamized roadbed, well fitted for the present heavy traffic, since the latter is limited to a few weeks in the summer.

No similar systematic improvement has been made in the numerous bridges along the road, although many of the bridges are quite old and their reconstruction, from the standpoint of safety, is of even greater importance. Near its upper end, this road crosses the Ausable River on a 75-ft. pin-connected steel span, about 30 years old, which has very light sections and poor details. The bridge would hardly be fit to carry present-day traffic, even if it were in perfect condition. Five or six years ago, one of the abutments was undermined by a flood, allowing the bridge to drop about 15 ft. on the rock bottom. The steelwork was badly injured, but was straightened as well as possible and re-erected on the new masonry. It is, however, in bad condition. The eyebars, in many panels, are so bent that one of the two bars carries the whole stress; but, worse still, the trusses are badly out of plumb and twisted, the top of one end post overhanging by as much as 12 in. in a height of 8 ft. As the bridge is a pony span with no lateral top bracing and no sway bracing to keep the top chords from buckling sidewise, the latter form a very crooked compression member nearly 80 ft. long under a fairly heavy stress. A truss in this condition, of course, would be very unsafe, even if the several members had ample cross-sections.

The bridge has remained about as described for the past five years. About three years ago, its condition was pointed out to many prominent residents and summer visitors, who after a moment's inspection seemed, without exception, satisfied that "something should be done at once". However, apart from fastening one of the end posts to a tree with a piece of telegraph wire, no repairs as yet have been made. As a matter of fact, the span could be made reasonably safe for $750, but it is doubtful whether public opinion will be intelligent enough and strong enough to insist on any action in the matter.

This incident would hardly be worthy of mention, but for the fact that in New York State at least there is no means of getting a bridge of this kind repaired except through the pressure of public opinion. A former law instructed the State Engineer to have highway bridges which are reported as dangerous, inspected by his staff and, if unsafe, closed to traffic. It is understood that this law was repealed some years ago through the influence of country members of the Legislature to avoid expensive bridge repairs. Outside of the larger cities, the matter rests entirely with local bodies.

It is understood that, in many other States, conditions are better, but, on the average, highway bridges the country over are weak under modern loads. Conditions have changed; a generation ago the traffic consisted mainly of light vehicles belonging to local residents. At present, heavy automobiles go even into remote districts. The older bridges were suitable for carrying the lighter loads. It is hardly fair, and certainly not practicable, to expect the poorer country districts to stand the expense of renewing immediately, all bridges not fully up to modern standards. They are, on the whole, doing as much as can be expected in replacing the old spans with stronger ones as

needed. To eliminate all dangerous structures within a reasonable time, it seems equitable that the cost should fall, in large part, on a more extended political district than the town or county. The inspection of existing bridges at proper intervals and the approval of plans for new ones should come under some proper State authority.

H. W. Brown,* Jun. Am. Soc. C. E.—Several of the speakers have emphasized the need for training inspectors and others connected with highway work. Some have suggested that European schools, or methods, trained a man better than American schools or methods. One discussor has even gone so far as to state that, of all the highways he saw in France, not one was incorrectly located.

It was the speaker's good fortune while with the Army in France, to attend L'Ecole Nationale des Ponts et Chaussées, otherwise known as the National School of Bridges and Highways, for a few months after the Armistice. Even though a graduate of, and, later, an instructor in, one of the foremost schools of the United States, the speaker found that he had much to learn, both in technique and in methods of teaching, from the French system. Perhaps a brief résumé of his observations of the working of this system will not be out of place here.

The principal engineering work in France is controlled by a corps known as "Ingénieurs des Ponts et Chaussées". These engineers are chosen from the highest graduates of L'Ecole Polytechnique, a school which corresponds somewhat to American colleges and scientific schools.

With the rank of "Elèves Ingénieurs", these men are sent by the Government to L'Ecole des Ponts et Chaussées for two years. The instruction at this school is given by men high in the active practice of their particular branch of the profession, and amounts to considerably more than that given at the average American school, because an attempt is made to start the new students with the benefit of the experience of the older men in analyzing the general, as well as the specific, elements of the problems which they will be likely to encounter. Little attention is paid to the method of analyzing stresses and strains—the students are well trained in mathematics and mechanics before entering the higher courses—but great stress is laid on methods which many practicing engineers in the United States would class among their professional secrets—things that it is claimed can only be learned by experience. The result is a thoroughly educated engineer who starts out with the combined experience of his predecessors as his foundation.

The uniform standard of excellence which these engineers have been able to maintain in all their work, has won for them great public esteem and confidence. That this confidence is richly deserved, has been implied by at least one of the speakers, and it is the consensus of opinion of the majority of careful observers of French engineering practice.

These few points are mentioned because of the feeling that Americans, as engineers, ought to investigate the work which other nations have done, in order to better their own methods. Surely engineers with the experience of the men who have discussed this subject, could investigate the methods the

* Engr., The Housing Co., Boston, Mass.

French have been developing since the time of Louis XIV, to help Americans solve in the best possible manner their problem.

M. S. Ketchum,* M. Am. Soc. C. E.—In 1921, the Civil Engineering Department of the University of Pennsylvania, in co-operation with the State highway departments of adjacent States and the Federal Bureau of Roads, gave a three weeks' course in Highway Engineering. The course consisted of lectures, recitations, and laboratory work. The instruction in laboratory work was given by the Staff of the Civil Engineering Department, assisted by engineers from practice who had had a large experience. The course was attended by 67 men, varying in age from 19 to 59 years, with an average age of 30 years. All the men attending the course had had extensive experience in highway engineering and many of them were technical graduates.

The second session of this course was given from January 23d to February 10th, 1922. In addition to the subjects offered in 1921, work was given in Contracts and Specifications and in Maintenance of Highways. Mr. Hilts had charge of the subject of Contracts and Specifications in which particular attention was given to the duties and responsibilities of an engineer in charge of work.

The brief course in Engineering, in which the instruction is given by engineer specialists in various lines, offers an opportunity for practicing engineers to keep up with modern developments in design and construction. It also offers an opportunity for the practicing engineer to give material assistance in engineering education, and will also bring about an active co-operation between engineers in practice and teachers in engineering schools. This co-operation should be of great value in developing the laboratories and arranging the various courses of study. Large business organizations are founded on much the same lines as educational institutions. The manager or chief engineer is practically the president of the educational institution; the assistant engineers are instructors, and the inspectors and foremen are assistants. The idea of the organization is to educate all the men in the organization, including the common workmen. The engineer and the contractor, instead of working at cross purposes, are in active co-operation. With this modern organization, the technical graduate is much more serviceable than he was a few years ago. Engineering graduates have always been criticized; they always will be criticized. They are young; they have the faults of youth, but the fortunate thing about young men is that if one gives them time, they will always get over their youth.

W. K. Hatt,† M. Am. Soc. C. E. (by letter).—The main problem of highway transportation is in the field of economics. Since a limited amount of the National wealth is available for transportation, what is spent on highway transportation beyond the economic necessity will diminish the amounts that should be applied to other agencies of transportation, such as railways and electric lines. The central need, therefore, is the organization of the transpor-

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tation agencies of the United States so that each may preserve its proper economic relation to the others. The data on which the solution of this problem may be predicated are largely wanting. Such data result from traffic surveys and analysis of data of costs that arise from the road and from the vehicle. It is in this economic field that the greatest opportunity lies for useful and fruitful research. Scientific correlation of transportation agencies should replace conflict.

It is well known that in industrial regions, like the State of Connecticut, the railroads have been largely relieved of their less than carload business which is now carried by motor trucks over the highways. It is reported that between Bakersfield and Los Angeles, Calif., 63% of the freight finds its way over the highway, at a cost of 18 cents per ton-mile as against 5 cents per ton-mile on the railway. Of course, speed of delivery and absence of terminal charges are the deciding factors in such traffic. In the vicinity of Seattle, Wash., it is said that the development of motor transportation companies has diminished the need of the farmer for his own truck. His products are placed on a platform in front of his farm and are delivered by a transportation company to the city, and the commodity that he desires is returned to him. The economic radius of such highway traffic is variously fixed at from 50 to 100 miles, depending on conditions, but the real economic radius will be affected by distribution to the truck, of costs that are not in the present balance sheet. That this truck transportation is here to stay cannot be doubted for a moment.

From 1910 to 1921, motor vehicles increased in number from 501 000 to 9 750 000, that is, 1 800%, and trucks increased from 2.8% to 10.3% of total vehicles. At present, there are nearly 8 500 000 automobiles and 1 333 000 trucks.

Preliminary figures indicate the investment in highway transport to be $20 000 000 000 outside of cities, including vehicles, garages, and roads, an investment equal to that in rail transportation. The motor vehicles represent an investment of $10 000 000 000. Nearly one-third of the vehicles are owned by farmers. There are 20 000 motor buses in operation; 12 000 schools are using motor buses for transporting children. Motor express lines in operation number 1 500, and 6 000 000 head of live stock were transported by motor truck in 1921. In Connecticut, a truck load of silk products valued at $100 000 passed over the roads. In Iowa, there is 1 car to every 4.5 persons; on the average, there is 1 car to every 10 persons in the United States.

The total operating expenses of motor vehicle traffic—passenger and freight—in the whole United States are so large that the fixed charges from capital costs and the additional maintenance costs arising from the road appear to be of less importance in relation to the total cost of transportation than would be supposed. Using data which represent reasonable estimates rather than exact statistics, one finds the following:

During 1921, the money spent on highways in the United States for State, county, and other roads outside of cities, was: Construction, $420 000 000; maintenance, $180 000 000; total, $600 000 000.

There were 9 750 000 vehicle licenses, that is, 8 404 000 for automobiles and 1 346 000 for trucks. If the cost of the road is divided by the number of

vehicle licenses, it appears that the total road expenditures for a licensed car are, as follows: Spent on maintenance per license, $18.60; total expenditures per car license, $61.50. Fixed charges arising from previous construction are not included.

Turning to the other side, one may ask what revenue was received from the licensed vehicles. During 1921, approximately $118 000 000 were paid for vehicle licenses in the United States, and approximately $120 000 000 of excise taxes, the latter of which did not go directly back to the roads, or a total of $238 000 000.

In a similar manner, one may divide the revenues received by the number of licenses, and find that the revenue per car for license fees was approximately $12 per year, and that adding the excise tax the total revenue from the car is approximately $19.

In other words, the car licenses paid about two-thirds of the cost of maintenance of roads. Of course, this showing is likely to be misleading because some money is spent on roads over which there is little automobile travel, and then, again, not all the money paid for license fees is returned to the roads. In addition, license fees are received from vehicles that travel almost entirely on city streets. One must also remember that large sums of money are spent by counties and townships in excess of revenues received by the State for license fees.

It is difficult to examine the calculation for individual States on account of diversity of practice in financing. However, proceeding as indicated, for the States of Iowa, Michigan, New Jersey, New York, and Pennsylvania, it is found that in 1920 the license fee per car was from $13.10 to $17.00, or an average of $14.53, and that the road expenditure, including construction costs for 1920, and maintenance costs, runs from $29.15 to $69.00, an average of $47.65.

Of course, the total highway expense for 1920 would include the fixed charges arising from the construction of preceding years. An effort is now being made by the U. S. Bureau of Public Roads to determine these costs for the entire United States.

As to the ability of the traffic to carry this burden, it may be said that the road licenses at present are probably not more than 2% of the total cost of the transportation, that is, the truck owner or the automobile owner pays in road fees only about $2 out of every $100 of fixed charges and operating expenses. Therefore, if this $2 is increased to $4 for licenses or road fees, the total cost of transportation would be increased only by 2%, so that when the farming communities and other property holders ask that road construction be stopped in order that road taxes may be diminished, it would appear that the road programs might still proceed, if only those who use the roads are asked to pay for them in license fees and gasoline tax, or other method. This statement is made merely for the purpose of stressing the importance of the traffic and not as an argument for such a method of financing.

A sound theory of highway finance is needed to determine the distribution of the cost according to benefits. In this theory, the various elements of the

road, such as right of way, grading, and structures, and the wearing surface, should be recognized.

Finally, it must be said that the maintenance work on the roads is not sufficient at present and should be increased.

EDWARD W. BUSH,* M. AM. SOC. C. E. (by letter).—There is need of a clearer understanding among engineers and contractors of the functions of a surety bond, the elements of suretyship, and the requirements for obtaining bonds. The subject is of importance to contractors because practically all public construction contracts are required by law to be protected by adequate surety bonds. The subject should also interest engineers as they outline the conditions under which the work is performed, and it sometimes happens that they unknowingly insert conditions that make it difficult for contractors to obtain bonds. Experienced contractors always arrange with sureties to write their bonds before their bids are filed, and anything that restricts the bidding of reputable contractors or limits free competition in bidding is uneconomic and, therefore, against the best interests of those engaged in the construction business. The writer, as the engineer of a surety company doing an extensive contract bond business, has an exceptional opportunity to note why it is often difficult to authorize bonds for contractors and, also, how some engineers and contractors misunderstand the relations that exist between the contractor and his surety. The following comments are offered as the opinions of an engineer who did not enter the surety business until after many years of active practice in the design and construction of engineering work. His former opinions were materially changed after acquiring knowledge of the business and the liabilities assumed by a surety in writing a bond.

Surety Bonds are a Form of Credit, and not Liability Insurance.—Surety bonds are a form of credit and not a form of liability insurance. They are a form of credit because they increase the responsibility of the contractor to the owner by the amount of the bond. They are underwritten by the surety on much the same basis that a bank extends a loan to the contractor, and for the same reasons. Both sell the contractor some of their financial strength for a fee, but there is always this great difference, the bank makes a short-time loan and always desires to be in a position to call its loan if the contractor is failing, whereas the surety cannot recall its bond, but having joined the enterprise, must continue to the bitter end, if it turns out that way.

The fact that a surety bond is a form of credit is the nub of this whole question. A bank meets with losses, but a loan is never made if the bank officials think there will be one; in the same way, a surety will never furnish a bond if it is thought that there will be a loss—the fee is not large enough to consider the business on any other basis. If the bank or the surety sustains a loss, they simply were misinformed or bad judgment was used in underwriting the risk.

The Underwriting Rule that Fixes the Amount of Bonding Credit.—Accepting the foregoing as true, it is readily understood that there is a limit to the amount of bonding credit of every contractor. Other things being equal, and

* Engr., Aetna Casualty & Surety Co., Hartford, Conn.

assuming that the contractor is honest, experienced in the kind of work considered and has adequate plant and organization, this limit is fixed by the relationship existing between (*a*) the contractor's net quick assets; (*b*) total net worth; and (*c*) the volume of work on hand. These are the measures by which is determined the amount of bonding credit the contractor can obtain from a reputable surety company.

Mr. St. John states* that for simple non-hazardous contracts, the sureties like to see the net quick assets, (*a*), at least 10%, whereas on extra hazardous classes of work, they desire the contractor to possess a net quick asset that might amount to as much as 20% of the total work to be performed. Mr. St. John did not comment on (*b*), the total net worth, but many sureties think that on ordinary classes of work it should be about 20% of the total work. Regarding the volume of work to be performed, (*c*), this is determined by adding the uncompleted part of the contracts on hand, expressed in dollars, to the contract price of the new work to be bonded.

Summing up, it is seen that the sureties have found, by past experience that on work of ordinary hazards it is desirable for the contractor to possess net quick assets of at least 10% of the total work he will have on hand, and that the total worth of the contractor should be not less than 20% of the work on hand.

Border Line Cases, and Re-insurance.—Contractors and engineers frequently do not understand why a surety will write several bonds for a contractor and then, perhaps, be lukewarm about assuming an additional risk. The contractor is probably assuming more work than his resources warrant and the risk becomes, what the surety calls, a "border line" case, and it may be that the bond can only be authorized after adequate re-insurance is obtained from other sureties or outside indemnity is offered by the contractor. Sureties freely re-insure their large risks and also their "border line" risks. Such re-insurance is not a reflection on the contractor's ability or standing; it merely means that the surety does not desire to carry the entire risk on the particular bond considered and, if re-insurance is obtained, it is in a position to go farther for the contractor and give him better bonding service. Re-insurance has benefited the contracting business, as many young contractors have been able to get a start because their risk was re-insured. The big contractors also are able to obtain bonds on the large jobs as well as to conduct a larger volume of business at any one time. Contracting is a hazardous occupation, and experience has shown that big contractors are not made overnight and those who eventually succeed do not take too much work for their resources, but become big through a natural growth. A surety that writes a bond on a contract that overloads a contractor is not doing the contractor a favor.

Sureties Decline Some Good Business Because Information Is Lacking.—As Mr. St. John has stated, sureties make mistakes in underwriting, and some bonds are written that never should have been, also they refuse some bonds to worthy contractors, but the writer knows that all sureties desire to write every bond that they can, and if a bond is refused it is because the surety did not have sufficient information to authorize it, the financial strength was not adequate, or it had adverse information concerning the contractor.

* See p. 1053.

Eliminating the Irresponsible Bidder.—Reputable contractors and engineers would like the irresponsible bidder eliminated from the construction business. He will be quickly eliminated as soon as contractors encourage the sureties to make full investigations prior to authorizing the bonds and supply the sureties with full information regarding their affairs. The reputable contractors can stand the investigation whereas the others can not. Engineers can assist by writing their advertisement so that a clear idea is obtained by the prospective bidder of the extent and the kind of work, by allowing plenty of time before bids are received, and by supplying the contractor, prior to the bidding, with additional copies of the specifications, contract, bond, and such sheets of the drawings as will give a general idea of the work for the use of the surety. It seems only fair to tell the surety what the obligations are, before it is committed to the suretyship.

Better Advertisements Will Attract Bidders, also Reduce Overhead Expenses.—The writer suggests that a radical change from the usual forms of advertisement is desirable, as most of them include many general conditions, that might better be included in a set of instructions to bidders. The advertisement should be worded so that the prospective bidder will be able to determine without a visit to the site whether his plant, organization, and financial strength fit the job and thus make it attractive on which to bid. It is to the advantage of every engineer to help reduce the general overhead expenses of contractors.

Requirements to Obtain a Contract Bond.—The requirements to obtain a contract bond are simple and become complex or mystifying only when the applicant forgets the credit relationship that exists between the applicant and the surety. Four things are required, namely, general information about the contractor, an accurate understanding of his financial condition, information on the amount and status of other work on hand, and, information about the proposed work. These points have been presented by Messrs. St. John and Lunt and the writer desires only to add that sureties have trouble in analyzing the financial statements of contractors and in distinguishing between the actual earned accounts receivable and mere bookkeeping accounts receivable, that may or may not include anticipated profits. Sureties, like banks and other organizations dealing with credits, allow no items that are not obvious or explained, and it is hoped that the contractors, through their associations, will adopt a standard form of financial statement that will show in detail just how the statement was prepared. At present, the greatest variation exists in the manner in which accounts are kept or assembled by contractors in preparing statements.

Many Losses Paid without Publicity and before the Contract Is Defaulted.—The premiums collected on surety bonds look like "easy money" to many engineers and contractors because they do not realize the extent of the losses paid by surety companies, as a large proportion of the claims are paid without publicity. It frequently happens that a contractor gets into financial difficulty and his bank will no longer extend him credit; the material men are pressing their bills and the payrolls must be met, or the job will shut down; the contractor will approach his surety (and very properly, too), explain his difficulty, and ask

for help. The surety, after making a rapid investigation and realizing that it is to its advantage to keep the contractor going, will extend the assistance that is needed to complete the contract and discharge the obligations of suretyship, but the fact that assistance was given may be known only to the contractor, his bank, and his surety. In a case of this kind, there is no default by the contractor, whose reputation is saved, and he is in a fair way to continue in the business of contracting and thus rehabilitate himself.

Rates Cannot Drop Until Losses Are Reduced.—Mr. St. John has mentioned the high mortality among surety companies. The writer has been told by an authority that during the past fifteen or twenty years from thirty-five to forty companies have quit the bonding business, either voluntarily or because of a forced liquidation. Some believe that the present bonding rates are too high, but the past history of the business seems to indicate that the rates cannot be materially lowered until the losses are reduced.

Principal Causes That Produce Losses on Contract Bonds.—The writer recently made a study of the claim files of his company to determine the principal causes that produce losses to sureties on contract bonds. The causes are:

(1).—Bid prices not adequate. It might be said that this is the sole cause of contract bond losses, as any work can be completed at a profit if the price is high enough, but it is not considered that way, only in the ordinary sense. It is well to keep in mind that what is adequate for one contractor, because of his superior ability, organization, and plant, is inadequate for another. The adequacy will often depend on whether or not the contractor will personally and closely supervise his work.

(2).—Increase in labor costs, or decrease in labor efficiency from the standards bid on. This also involves the scarcity of labor.

(3) Failure to obtain materials and supplies as expected, and for the prices considered. After a sand or gravel bank, or a stone quarry, is opened up it may be found that the quality of the material does not meet the requirements of the specifications. The engineer is sorry, but the contractor is the one who pays the additional cost of supplying the better material.

The writer believes it will pay engineers to test the local sources of materials and pass on their acceptability before bids are received. The engineer has to pass on them some time, and if he does so before the bids are submitted, he removes an uncertainty regarding the cost of the work, and every time an uncertainty is removed, the cost is lowered. Contracts for materials are often broken and must be re-let at advanced prices; railroad transportation is not always available; freight rates change; the State may supply part of the material and be slow in deliveries; etc. There are many ways in which materials may cost more than as first considered.

(4).—Misrepresentation of financial condition of the applicant. Every surety pays claims because the true financial condition was not disclosed when the risk was accepted.

(5).—Misrepresentation of amount and status of other work on hand. Contractors sometimes have contracts under way that are being done at a loss,

and they will endeavor to take another contract with the hope that it will pull the others "out of the hole".

(6).—Inability to consummate the financial arrangements planned when the work was taken. The nature of the business requires that contractors become frequent borrowers at banks, but the contractor cannot get funds if they are not available. In many parts of the United States, contractors receive no payments on the contract until it is completed when they are paid in bonds, certificates, or warrants. In such cases, the contractor must conclude a binding arrangement with some banker to discount this paper and to advance funds with which the work is done. These bankers sometimes fail to meet their obligations.

(7).—Bad weather. This has caused many contractors to fail as the overhead consumes the profits and slow progress is also made on certain work days.

(8).—The default of sub-contractors is a contributing cause to many failures.

(9).—Assuming work the size or character of which is not justified by the contractor's plant, organization, and financial strength.

(10).—Difficulties arising that were unforeseen at the time of bidding. Every engineer and contractor can recall difficulties that were not suspected when the job was bid on, that have greatly increased the cost.

No Substitute for Corporate Surety Bonds.—There is no substitute for corporate surety bonds, their use was justified many years ago; their cost is reasonable for the protection they give, and they are an economic safeguard to the expenditure of public money. If the laws were changed so that they would not be required, a chaotic, disastrous condition would arise in the construction business and much of its present advanced position would be lost.

Two Kinds of Suretyship—Personal and Corporate.—There are two kinds of suretyship—personal and corporate. The first kind in general has proven unsatisfactory, because no individual will willingly pay a loss and if trouble arises, he is likely to employ a smart lawyer to find a loophole by which he can avoid his obligation. That is human nature.

The reputable corporate surety is regularly organized to pay losses, in fact, it can only continue in the business by cheerfully meeting its just obligations. Many years ago, some of the companies in the surety business were somewhat disposed to stand on technicalities, and trouble on a contract often meant a lawsuit with the surety. Those conditions no longer exist with reputable sureties and lawsuits to recover on a surety bond are seldom brought.

Premium is Based on Contract Price.—The sureties have found that the amount of the bonding risk or the hazards of suretyship are measured by the volume of the work performed rather than by the size of the bond required. The underwriting rule, used to determine the amount of bonding credit, is in accord with this.

The amount of the premium collected is based, therefore, on the contract price as it is the best available measure of the volume of the work performed. The sureties are not given an opportunity to determine the amount of the bond—that is done by the owner—but the sureties know that on ordinary classes of work the average losses, if any, will not exceed the contract price

by, say, an advance of 25 to 40%; therefore, if the amount of the bond is 40% or more of the contract price, the bond is large enough to compel the surety to assume the full loss. On ordinary contracts, the surety is not especially concerned whether the amount of the bond is 50, 75, or 100% of the contract price, because the surety knows the bond is large enough to cover the average loss that may occur. The premium of the bond on construction contracts is based, therefore, on the contract price and not on the penalty of the bond, and rates are made accordingly.

Sureties Dislike Maintenance Guaranties on Roads and Pavements.—The building of roads and pavements seems to attract the inexperienced contractor —the work looks so easy. Many men start in the business, but not many remain after the first few years, unless they develop more than ordinary ability and apply the same kind of methods that succeed in other business, with the result that the sureties are generally unwilling to assume maintenance bonds except for strong and well established contractors. They do not know whether the contractor will be in business and able to take care of the maintenance repairs a few years hence.

Guaranties are Uneconomic, Unfair, and Reflect on the Engineer.—The writer believes that all road and pavement guaranties are uneconomic because they make it difficult for some contractors to obtain their bonds, therefore, they restrict the bidding, and they also hire the contractor to carry a risk that can be carried more cheaply by the owner. They are unfair because they seek to place a responsibility on the contractor that rightly belongs to some one else. The requiring of a maintenance guaranty seems to indicate one or both of two things: The street or road commissioners distrust their engineer and believe he does not know enough to design and supervise a good road or pavement; or the engineer doubts his own ability to design or supervise the work. Either case reflects on the professional status of the engineer.

If guaranties are considered necessary, they should be limited strictly to those against defective workmanship and materials; however, it will be difficult to collect on a guaranty of this kind unless the construction work was not supervised or inspected. If engineers and inspectors supervised the job, they probably passed on the materials and workmanship, and it will be difficult later to claim that such supervision was incompetent.

Frequently, the guaranty is a little broader than the kind just mentioned, because it also guarantees against defects arising from the use of the surface as a roadway. In other words, it makes the contractor guarantee to the owner that the engineer knows his business. The contractor does not hire the engineer and has no part in determining the design of the road or pavement, therefore, why hold him responsible for results? No one knows the traffic that roads will be called on to bear and a good design to-day may be hopelessly inadequate two or three years hence.

The most unjust form of guaranty makes the contractor maintain the pavement in perfect condition irrespective of the causes that produce the defects, or by specifically excluding certain causes and leaving the guaranty active on defects arising from all other causes. These guaranties frequently

specify that, at the end of the maintenance period, the pavement must be in perfect condition.

The writer has seen some contracts that specified a minimum kind of guaranty clause, and elsewhere included a clause, which appeared to have been drawn by an attorney, by which the contractor was liable for everything, regardless of what happened. The causes of a failure of a road are not easily determined, and although a contractor may make slight repairs rather than dispute the claim, he may with propriety take a different stand if the cost of the repairs runs to a considerable amount and the owner may have difficulty in proving that the contractor is liable under the terms of any guaranty. Reputable contractors dislike to go into Court and also avoid bidding in localities that have the reputation of trying to drive hard bargains with their contractors, places where they must perhaps sue in order to preserve their rights. Road guaranties are unfair, uneconomic and lead to disputes and therefore, should be abolished.

Conclusion.—In conclusion, the writer wishes to state that it is desirable for a contractor to establish relations with a good bank and a good surety and to take them into his confidence before undertaking a large contract. Both are deeply interested in the success of his business, and if they are of the right kind, their advice is not to be taken lightly. The confidential information in the files of a surety company affords a splendid opportunity to study the successes and failures in the business of contracting. The records show the many causes of failures and they also show the large number of men who, because of their courage, high integrity of character, skill in the business, and resourcefulness in overcoming difficulties, achieve a just reward in financial independence and the regard of their fellow men. Their business is an exacting one and they fully earn all the success they attain.

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SIPHON SPILLWAYS*

By G. F. Stickney,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles F. Ruff, Edward Wegmann, W. P. Creager, H. F. Dunham, Karl R. Kennison, Joel D. Justin, H. B. Muckleston, Charles G. Darrach, Fred A. Noetzli, Robert E. Horton, A. Griffin, E. S. Lindley, J. T. Noble Anderson, and G. F. Stickney.

Synopsis

This paper relates to the use of the siphon for discharging water and for regulating the water level at a dam, or in a canal. The different types of siphons are described, and the principle of their operation is given. Various siphon spillways that have been built and are in operation in the United States, are described, together with the conditions prevailing at each site, illustrating the uses for which the siphon may be adapted.

The siphon spillway is a device‡ for discharging water at a dam and was introduced in the United States by the writer, on the New York State Barge Canal, thirteen years ago or more. Since the first spillways were constructed, numerous experiments have been made, and the siphon has been improved and developed, so that now it is used for discharging volumes of water far greater than was originally contemplated. The water is discharged through a closed conduit, which makes it possible to utilize the fall at a dam to increase the velocity of flow, and the spillway is put in operation by a slight rise in the water level, above the dam. The action is entirely automatic and priming takes place quickly, so that the full capacity of the spillway is effective at once. The fluctuation of the water surface, above the normal stage, is restricted within narrow limits. The rate of discharge over a dam depends on the depth of water on the crest, increasing as the depth increases, and hence an overfall spillway may not have much discharge capacity until the water sur-

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† Cons. Engr., Albany, N. Y.

‡ Patent No. 991,907, filed March 20th, 1911, Serial No. 615 736.

face rises to a considerable height. Such a spillway is not immediately responsive to variations in the stream flow and does not closely control the water level. Properly designed siphons have a wide range of usefulness in hydraulic developments of various kinds.

Although siphons were known to the ancients and have been utilized for hundreds of years, their use was confined to small tubes, of an inverted U-form, for conveying liquid over the edge of a vessel and delivering it at a lower level; it is within recent times, only, that the siphonic principle has been applied to conduits of large size and capacity, for discharging water at a dam. The action depends on the difference in pressure at the extremities of the conduit, the flow being toward the lower level and ceasing when the levels coincide, or when air collects at the highest part of the conduit. Water flows, by gravity, from a pool or basin to a lower level, and this flow is produced by a column of water the height of which is equal to the difference in level of the free water surfaces at the inlet and the outlet. In order to pass water over a summit in the siphon, higher than the upper water surface, it is necessary to utilize the air pressure on the water surface. A clear idea of siphonic action may be had if it is understood that the flow is due to the "push" of air on the upper water surface and in no way is due to any "pull" of water in the lower leg. This fact being grasped, it is evident that there is no possibility of increasing the flow through a siphon by prolonging the lower leg below the siphonic limit.

Normal atmospheric pressure at sea level, is 14.7 lb. per sq. in. at 32° Fahr., which is equivalent to the pressure at the foot of a column of water 33.9 ft. high, and is the greatest head that may be utilized for producing siphonic flow. The decrease in atmospheric pressure may be taken as 0.5 lb. for each 1 000 ft. above sea level, hence the siphonic head decreases, approximately, 1 ft. for each 850 ft. of altitude.

Consider a siphon with its inlet and its outlet submerged, as shown in Fig. 1. The water surfaces are represented, by dotted lines, as being raised above their true levels to such heights as would be necessary to give pressures equal to the atmospheric pressure at the ends of the siphon. The water at the inlet, being under pressure, would completely fill the conduit and one can imagine the siphon as being deeply submerged. All that is necessary, to bring about such a condition, is to exhaust the air from the siphon, and rapid flow will take place. In order that a siphon may flow continuously, it is necessary that all parts shall lie below the hydraulic grade between the raised upper and lower levels, as shown, since if any part is above such an hydraulic grade, air will accumulate and will cause a back pressure against the water at the inlet that will stop the flow. A siphon that lies well below the hydraulic grade may receive a considerable volume of air without effect other than to lower the grade which, of course, decreases the discharge. In such case, the air is carried through the siphon, along with the water, and passes out as rapidly as it enters. If, however, more air is admitted than the water can carry, the hydraulic grade drops and the flow ceases. A siphon in operation, and acting under the siphonic head or less, will continue in operation as long as the inlet is submerged sufficiently to prevent the entrance of air, whether

or not the outlet is submerged. This is due to the water flowing at too high a velocity for air bubbles to rise in the lower leg against the outflow. The problem is thus resolved into the devising of methods for priming the siphon. A siphon that is sealed against the entrance of air, will be primed when there is sufficient flow through the conduit to absorb and carry out the air entrapped therein. In some siphons, it is necessary that the lower leg be sealed by discharging below the lower water surface or by a sealing basin at the end of the leg, whereas others discharge freely and are sealed by a jet of water in the siphon. The inlet of the siphon should be large, in order that the velocity of inflow will be small and thereby reduce the loss of head at entry. The inlet should also be placed well below the water surface, to exclude float-

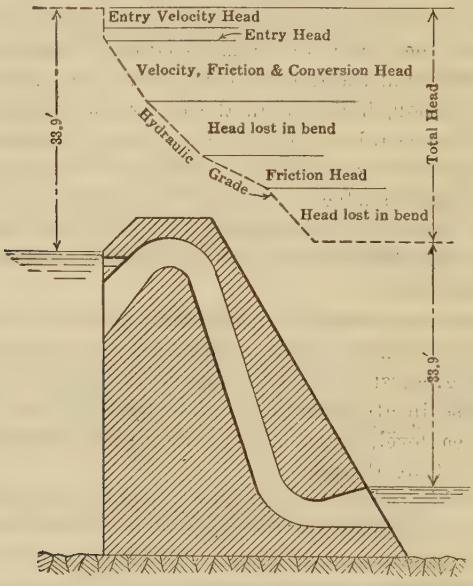


FIG. 1.

ing matter which might clog the conduit. The upper leg should taper, gradually, from the inlet to the minimum section, called the throat, at the top of the lower leg. The upper bend, forming the crown, should be made with regular curves of moderate radii. The lower leg may be either vertical or inclined and, ordinarily, should be of uniform area throughout; the latter, however, is not strictly necessary. The outlet should be flared upward, so that air bubbles rising through the water while the siphon is priming, may be deflected outward. The air vent, extending from the up-stream face of the dam to the crown of the siphon, should be located at the normal water level and the crown of the siphon should be just above the normal level so that no water will spill until the water surface rises.

Let a siphon with a vertical lower leg be considered, as shown in Fig. 2. The water rises above the air vent, sealing it against the admission of air, and spills through the crown of the siphon, forming a jet at the throat that jumps across the lower leg. This jet makes a diaphragm of water which seals

the upper part of the siphon against the entrance of air from below. A certain volume of air is entrapped above the jet in the crown of the siphon and this air is absorbed and carried out by the flowing water. It is not necessary to eliminate all the air to induce priming; the removal of a small part is sufficient to start the action. This is due to the fact that the pressure of a confined body of air varies inversely as its volume, and when a part is removed the remainder expands, losing density and pressure in the process. A loss of 3% of the volume of air in the upper part of the siphon will cause the water to rise 1 ft. in the crown and will give a substantial flow through the conduit. As the flow increases, the capacity of the water for absorbing air increases and, in a brief interval of time, sufficient air will be removed to cause the water to fill the crown of the siphon and then the air, in the lower leg, will be driven out. Priming takes place quickly, after the flow through the siphon is sufficient to form a diaphragm of ample thickness to exclude air, and requires only a few seconds. When the water level above the spillway is drawn down so as to unseal the vent, air rushes into the crown of the siphon in such volume that the siphonic action is broken and the flow abruptly ceases.

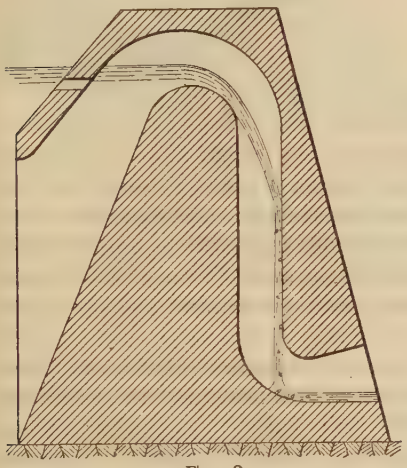


FIG. 2.

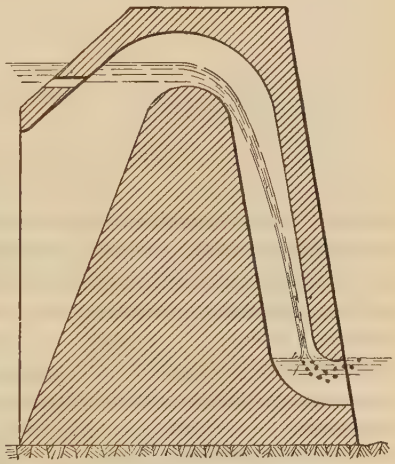


FIG. 3

A siphon with slightly inclined lower leg and a sealing basin at its bottom, is shown in Fig. 3. A rise in the water level above the dam seals the upper end of the siphon and produces a flow of water through the conduit which fills the sealing basin and thus seals the lower end. A jet of water is formed at the crown of the siphon, which plunges into the water of the sealing basin, carrying air absorbed from above, and this air is carried outside the conduit before the bubbles can rise through the water. As in the previous case, priming is a progressive process and is quickly accomplished.

A siphon with the lower leg on a flatter slope and also terminating in a sealing basin, is shown in Fig. 4. In this case, the inclination of the lower leg does not admit of a jet being formed directly, and in the crown, and to the ends and to the far side of the lower leg, recourse is had to an auxiliary chan-

nel where jets are formed, that seal the upper part of the siphon. This causes the upper part of the siphon to prime and produces a flow in the lower leg which carries out the air and primes the lower portion. The sealing basins are provided with small pipes to drain them of water when the siphons are not in operation, so that they will not become clogged with ice in cold weather.

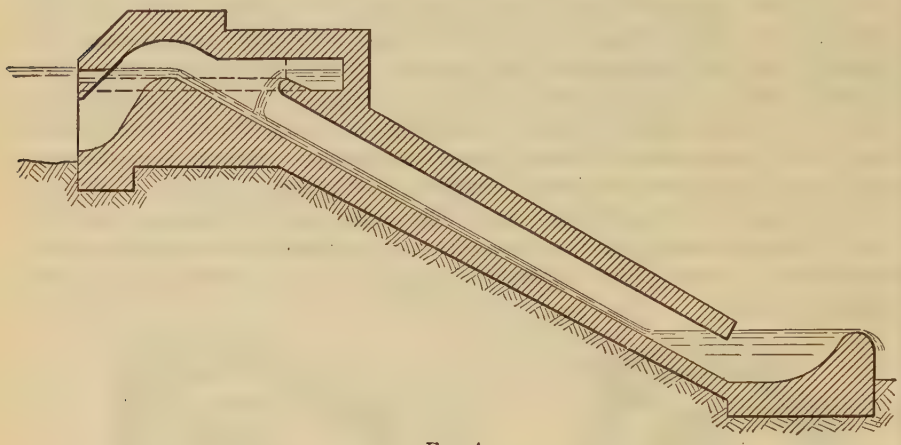


FIG. 4.

Another type of siphon, with inclined lower leg, which does not require a sealing basin, is shown in Fig. 5. In this siphon, a curve is formed in the plane of the back wall of the lower leg, near the bottom, such that the water flowing through the conduit will be deflected and a jet will be formed that will jump across the leg. This jet makes a diaphragm of water which seals the leg against the entrance of air from below and causes the siphon to prime.

A siphon primed by an auxiliary device is illustrated in Fig. 6. The crown of the main siphon is entirely above the water level, and the lower leg terminates in a sealing basin. Beneath the main siphon is a smaller siphon that has a perforated, horizontal pipe extending along its throat. This pipe is connected with the crown of the main siphon by branch pipes. The priming siphon, with its crown just above the normal water level, is put in operation by a slight rise in the pond above the spillway and the rapid flow of water over the perforated pipe draws air out of the main siphon. The removal of a small part of the air, entrapped in the large siphon, causes the water to rise and flow through the crown of the latter and this produces a jet that completes the priming. This type of siphon is not as quick-acting, as those previously described, requiring a few minutes to get under way.

All the previously described siphons will prime when there is a slight rise in the water level above the dam, the rise varying from about 3 in. to 1.5 ft., depending on the size and type of the siphon.

Although some types of siphons require submerged outlets, sealing basins are objectionable and should be dispensed with, if possible. When water rises

in a sealing basin, the air entrapped in the siphon is compressed, which may, unless relieved, retard or prevent priming. Further, when priming starts there is a pulsation in the flow, due to air rising inside the siphon from the water of the sealing basin and restoring a part of the volume previously removed. The sealing basin reduces the effective head somewhat and requires considerable additional masonry, which increases the cost of the structure.

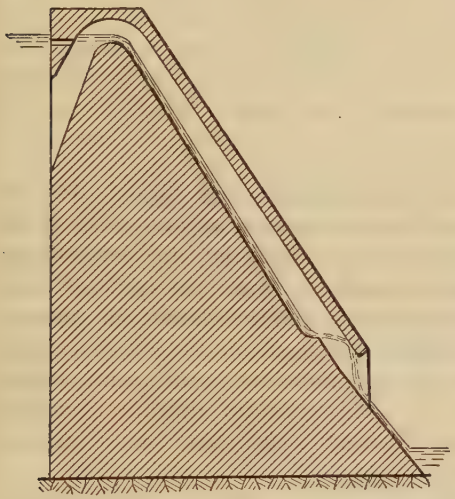


FIG. 5.

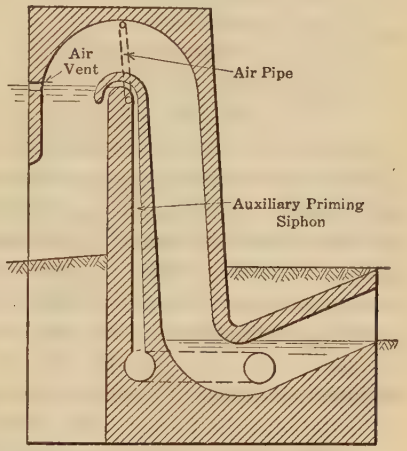


FIG. 6.

During operation, the shell of a siphon is under pressure, the intensity depending on the head. The air pressure tends to collapse the walls of the conduit, but this is offset, to some extent, by the fluid pressure within the siphon. In accordance with Bernouilli's theorem, the pressure head plus the velocity head, at any section of a tube under steady flow without friction, is equal to the static head at the section when there is no flow. From this, it follows that the internal pressure, at any point in a siphon, is equal to the pressure at the inlet less the losses in pressure that occur between the inlet and the point in question. The pressure at the inlet is the pressure of the atmosphere on the water surface, equivalent to the hydrostatic pressure from a head of 33.9 ft., plus the head of water over the inlet. The internal pressure at the crown of the siphon is a minimum and may be zero, if the siphon is acting under the maximum siphonic head, and in such case the unbalanced atmospheric pressure will amount to 2 117 lb. per sq. ft. The effective outside pressure decreases almost to zero at the inlet and at the outlet.

The discharge of a siphon may be expressed by the following formula:

$$Q = C\,a\,\sqrt{2\,g\,H}$$

in which,

Q = discharge, in cubic feet per second.
C = coefficient, varying from 0.6 to 0.8.
a = minimum cross-sectional area, in square feet.

H = head, in feet.
g = acceleration of gravity = 32.16.

The total head acting on a siphon is expended in producing velocity of flow and in overcoming the resistances due to entry, to conversion, to friction, and to bends. This may be expressed by the following formula:

$$H = h_v + h_e + h_c + h_f + h_b$$

in which,

h_v = velocity head.
h_e = entry head.
h_c = conversion head.
h_f = friction head.
h_b = head lost in bends.

It has not been found possible to derive a satisfactory formula for the flow through a siphon, that will take all these factors into account as there is, as far as the writer knows, no formula or experimental data for determining the loss of head at bends in rectangular conduits. From tests made of large siphons, it is found that the loss of head due to bends is large and approximates that due to velocity of flow. Fuller's experiments on the loss of head at bends in pipes indicate that this loss is a minimum for bends with a radius of from 4 to 7 ft., and this possibly also holds good for siphons. Properly designed siphons will have efficiencies of from 60 to 80%, but, for preliminary computations, the former figure should be used.

As the velocity of flow varies as the square root of the head, the discharge of a siphon does not change much for considerable variations in the head. This must be kept in mind and the spillway designed for the maximum flow to be expected. Siphons under low heads are effective, as may be realized by considering that the greatest proportional velocity of flow is due to the first part of the fall.

The first siphon spillway built in the United States is on Wood Creek, adjacent to Lock No. 12, at Whitehall, N. Y. Wood Creek which has a flow varying from 200 to 12 500 cu. ft. per sec., has been canalized, forming a part of the Champlain Canal. The surplus flow, not required for navigation, is used to develop power for a silk mill on the side of the stream opposite Lock No. 12. The fluctuation of the water surface during the navigation season is limited to 1 ft. above the low-water plane, which is closer regulation than is possible at this site with a fixed dam in the available width of 90 ft. A low dam was built here, with a movable crest of 8 ft., which may be raised above the water surface in time of flood. In order to avoid frequent manipulation of the crest for minor fluctuations in the flow, such as occur from day to day, a siphon spillway was constructed in the forebay of the silk mill. This spillway consists of six siphons each 0.5 ft. high by 8.6 ft. wide, and acting under a minimum head of 13 ft., has a capacity of about 450 cu. ft. per sec. The crown of the siphon is just above the low-water level and the inlet, 2 ft. high by 4.3 ft. wide, is 7 ft. below the water surface. The upper leg converges in section from inlet to throat, the latter being 4.3 sq. ft. in area. The lower leg is vertical and of the same area as the throat, but changes in

section from 0.5 ft. by 8.6 ft. to 2 ft. by 2.2 ft. at the outlet. Three 4 by 12-in. air vents, to the crown of the siphon, pierce the up-stream face of the dam at the low-water level and a fourth vent, of the same size, is located 3 ft. lower. A rise of 0.4 ft. in the water level is sufficient to start the siphons in operation. It was thought that the air vents might possibly become clogged, in which case the flow would not be stopped when the water was drawn down to them. To provide against such a contingency, a 3-in. pipe over the siphons connected with the crown of each and extended through the masonry, where the end was closed by a removable cap. It was believed that by removing the cap sufficient air would be admitted to stop the siphons. A test of this device resulted in failure to stop the siphons and the air admitted had no perceptible effect on their operation. This spillway has been in satisfactory operation since it was built and has never failed to operate during cold weather.

The next spillway is on the summit level of the Champlain Canal, at Lock No. 9, near Smith's Basin, N. Y. This spillway consists of four siphons, each 1 ft. high by 7.75 ft. wide, together with a waste weir, 20 ft. wide, which is provided to pass drift too large to go through the siphons. The capacity of this spillway, when there is a rise of 1 ft. in the water level of the canal, is about 555 cu. ft. per sec., the siphons passing 485 cu. ft. per sec. under a 10.5-ft. head. The siphons are similar in construction to those at Whitehall.

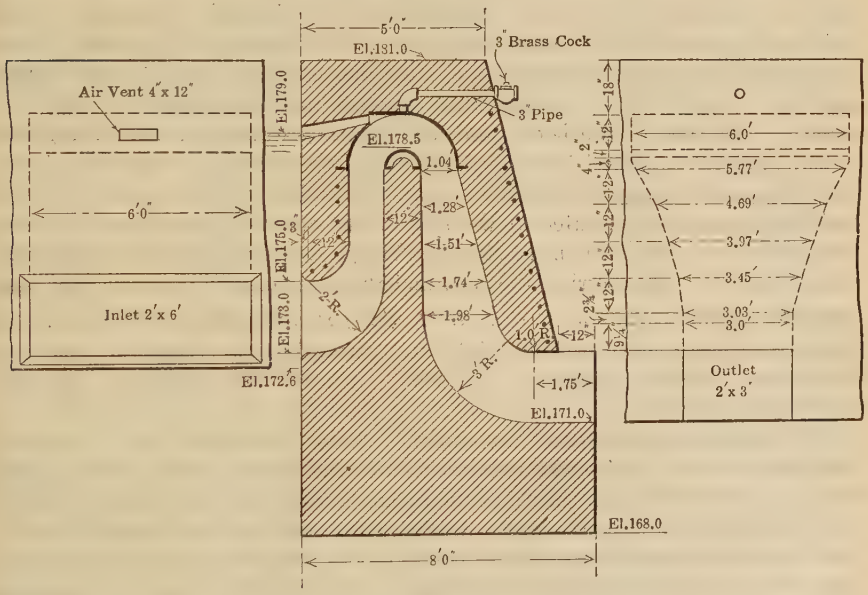


FIG. 7.

The siphon shown in Fig. 7 is on the Glens Falls Feeder which supplies water for the Champlain Canal. This feeder is a navigable channel with a drop of 134 ft. in a distance of 7 miles and extends from the Canal to the Hudson River above Glens Falls. There are 14 locks, either single or in flights,

around which the water is carried in concrete-lined by-passes and at the head of each by-pass is a concrete bulkhead containing four 2 by 3-ft. sluice-gates and a siphon spillway. There are eight of these bulkheads, identical in all particulars, with a single siphon, 1 ft. high by 6 ft. wide, which operates under a head of about 5.7 ft. The crown has a cast-iron lining which was used on account of the difficulty of removing forms from such a small conduit. The crest of the siphon is ½ ft. below the normal water level, so that at all times there is a slight flow through the spillway of about 8 cu. ft. per sec., which is not objectionable since there is a constant feed of water through the by-passes. A rise of 3 in. in the pool level submerges the air vent and the siphon primes immediately. A test was made to determine the discharge and the efficiency of the siphon at combined Locks Nos. 2 and 3. The by-pass is 16 ft. wide by 200 ft. long and has a fall of 12 ft., in four drops of 3 ft. each, with level stretches between. The drops are located 33 ft., 58 ft., 83 ft., and 108 ft., respectively, below the siphon outlet, with a fifth drop of 1 ft. at the end of the by-pass. A weir to measure the discharge was constructed at the lower end of the channel, 192 ft. below the siphon outlet. This weir was made of a 2-in. plank with an ⅛-in. steel plate, machined to a true edge, projecting above the wood on the up-stream side, and the edge of the plank was beveled on the down-stream side. Vertical planks at each end, with similar steel plates on their inner edges, reduced the length of the weir to 13.93 ft., and its crest was 0.95 ft. above the bottom of the channel. The head on the weir was measured by a bottle gauge in a stilling-box, at the side of the channel, 16 ft. above the weir. The gauge was made of a 1-gal. water bottle, used for a float, with a wooden staff, divided to hundredths of a foot, inserted in the neck of the bottle and held erect by guides on the stilling-box. Baffle-boards were placed in the by-pass above the weir to check the velocity and to equalize the flow of water across the width of the channel. A second bottle gauge was located at the side of the channel, just above the bulkhead, to measure the head on the siphon. The gauges were read simultaneously, at intervals of 1 min. during the test, and the discharge of the weir was computed by Bazin's formula, $Q = m\ L\ H\ \sqrt{2\ g\ H}$, using the coefficient, m, as determined for a weir 0.98 ft. high, and correcting the length of the weir by Francis' formula, making $L = 13.93 - 0.2\ H$. The results of the tests are shown in Table 1.

The cross-sectional area of the siphon is 6 sq. ft. and the inlet is 12 sq. ft. The head on the siphon was measured from the water surface to the upper tangent point of the outer curve at the siphon outlet, as it was found by reaching into the outlet, while the siphon was running, that the water was not in contact with the concrete below the tangent point and that air extended that far up the leg. The velocity head, entry head, and friction head may be computed from the test data and the sum of these, deducted from the total head, gives the loss of head due to the resistance at bends and to conversion in the upper leg, as shown in Table 2.

The siphon primed when the water surface rose 0.1 ft. above the top of the air vent and the siphonic action stopped when the water level was drawn

down to the top of the vent. A 3-in. pipe, closed by a valve, had been provided in the top of the siphon, in order to be able to stop the discharge in case the vent should be clogged, and when the valve was opened, air rushed into the siphon and the flow stopped.

TABLE 1.

Time, A. M.	Depth on weir, in feet.	Weir discharge, in cubic feet per second.	Head on siphon, in feet.	Velocity in siphon, in feet per second.	Efficiency of siphon, percentage.
11- 9	1.19	68.9	5.64	11.48	60.3
10	1.21	70.8	5.66	11.80	61.8
11	1.20	69.9	5.67	11.64	61.0
12	1.21	70.8	5.70	11.80	61.6
13	1.21	70.8	5.72	11.80	61.5
14	1.20	69.9	5.74	11.64	60.6
15	1.20	69.9	5.75	11.64	60.5
16	1.20	69.9	5.77	11.64	60.4
17	1.21	70.8	5.75	11.80	61.3
18	1.20	69.9	5.63	11.64	61.2
19	1.21	70.8	5.58	11.80	62.2
20	1.20	69.9	5.50	11.64	61.9
Average					61.19

TABLE 2.

Velocity in siphon, in feet per second.	Head on siphon, in feet.	Velocity head, in feet.	Entry head, in feet.	Friction head, in feet.	Other losses, in feet.
11.48	5.64	2.05	0.26	0.38	2.95
11.80	5.66	2.16	0.27	0.39	3.84
11.64	5.67	2.10	0.26	0.38	2.93
11.80	5.70	2.16	0.27	0.39	2.88
11.80	5.72	2.16	0.27	0.39	2.90
11.64	5.74	2.10	0.26	0.38	3.00
11.64	5.75	2.10	0.26	0.38	3.01
11.64	5.77	2.10	0.26	0.38	3.03
11.80	5.75	2.16	0.27	0.39	2.99
11.64	5.63	2.10	0.26	0.38	2.89
11.80	5.58	2.16	0.27	0.39	2.82
11.64	5.50	2.10	0.26	0.38	2.76

A siphon spillway is utilized at Plant No. 2 of the Tennessee Power Company on the Ocoee River, near Cleveland, Tenn. A timber flume with a capacity of 1 200 cu. ft. per sec. carries water to the plant, at which there is a head of about 250 ft. The flume passes through a small ravine high up on the hillside adjacent to the plant, where a forebay with a siphon spillway was constructed to discharge the flow in case of a sudden shut-down of the plant. The normal flow line is 1 ft. below the top of the flume and the area of the forebay is so small that the water would overtop the flume in a few seconds if a spillway was not provided. An overflow waste weir was impracticable and it was feared that mechanically operated gates could not be relied on to operate within the limited time available. A siphon spillway, occupying a length of 90 ft., solved the problem, both for rapidity of action and for capacity of discharge. This spillway contains eight siphons, each 1 ft. high

by 8 ft. wide. Half the units act under a head of 18.5 ft. and the other half under a head of 26.5 ft. and give a discharge of nearly 1 600 cu. ft. per sec. The siphons vary only in the length of the lower leg and are as shown in Fig. 8, which illustrates one of the longer siphons. On the completion of the plant, tests were made to determine the maximum rise of the water surface in the flume above the spillway, the speed of priming the siphons when the water rose at various rates, and the discharge capacity of the spillway. To make this test, the flow in the flume was adjusted to supply the turbines of

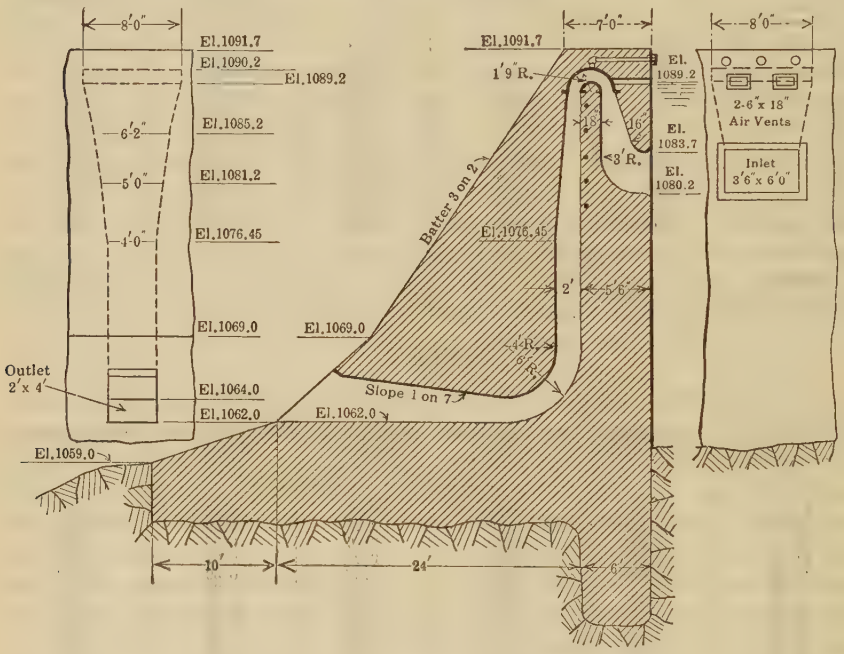


FIG. 8.

the plant and to maintain the water level in front of the spillway at the crest of the siphons (Elevation 1 089.2). By pulling a switch in the plant a predetermined load was thrown off the generators and the turbine governors closed or partly closed the wheel-gates, which caused a rise in the water level in the flume. A staff gauge, reading to hundredths of a foot, was established on the face of the spillway between Siphons Nos. 3 and 4, to give the elevation of the water surface, and time intervals were determined with a stopwatch. Six tests were made, from which the data in Table 3 are taken.

Before the rise began, the water was practically at Elevation 1 089.2 in each case, and the top of the air vent is at Elevation 1 089.45. Since priming could not occur until the air vent was submerged, the lowest elevation at which siphonic action could be established was 1 089.45. Since the discharge through each unit before the siphonic action was established would be small, it may be assumed that the water level rose at a uniform rate during each

test, although this is not strictly true, and the approximate time required to prime the siphons may be computed from Tests Nos. 2, 4, 5, and 6, as shown in Table 4.

TABLE 3.

Test number.	Power thrown off, in kilowatts.	Maximum elevation of water surface, in feet.	Time of rise, in seconds.
1	1 500	1 089.55	....
2	1 700	1 089.55	21
3	3 400	1 089.65	....
4	3 000	1 089.57	17
5	6 000	1 089.75	12
6	9 000	1 089.95	8

TABLE 4.

Test number.	Elevations of Water, in Feet.		Rise, in feet.	Time, in seconds.	Rate of rise, in feet per second.	Time Required, in Seconds.	
	Low.	High.				To seal.	To prime.
2	1 089.20	1 089.55	0.35	21	0.01667	15.00	6.00
4	"	1 089.57	0.37	17	0.02176	11.49	5.51
5	"	1 089.75	0.55	12	0.04583	5.45	6.55
6	"	1 089.95	0.75	8	0.09375	2.67	5.33

From Table 4, it appears that the time required to prime is probably a constant for any particular siphon and is not affected by the rapidity of the rise in the water level. Large siphons might be expected to require a longer period to prime, but the action is certainly very rapid for the type of siphon installed at Ocoee. The capacity of two siphons, Nos. 5 and 6, acting together, was determined in the following manner: The gates at the head of the flume were adjusted to give a flow of 390 cu. ft. per sec., as determined by a rating curve, and the air vents in Siphons Nos. 5 and 6 were sealed, so that they would continue to operate when the water was drawn below the normal level. The turbine gates were then closed, so that no water was drawn off through the plant and, in consequence, the water in the flume rose, putting all the siphons in operation. The spillway discharge, being much in excess of the flow in the flume, caused a quick lowering of the water level and when the air vents of the siphons were exposed, all units, except those with sealed vents, ceased to operate. This left Siphons Nos. 5 and 6 in operation, and as they were discharging more water than the flume supplied, the level continued to fall. After the water had been drawn down to Elevation 1 088.7 (0.5 ft. below the crests of the siphons), the time was observed. A second observation, made 7.5 min. later, showed a further drop to Elevation 1 088.4, or a fall of 0.3 ft. in 7.5 min. The average head on the siphons, during these 7.5 min., was 25.55 ft. (measured from the water surface to the center of the outlets) and the siphons discharged the 390 cu. ft. per sec. flowing in the flume plus a certain volume of water drawn from storage in the flume and the

forebay. The area of the forebay is 10 000 sq. ft. and the area of 300 lin. ft. of flume, beyond the spillway, is 4 260 sq. ft., making a total area of 14 260 sq. ft. The volume of water taken from this area in 7.5 min. was $14\,260 \times 0.3 = 4\,278$ cu. ft., which is equivalent to a flow of 9.5 cu. ft. per sec. In addition, a volume of water was drawn from the flume, between the spillway and the head-gates, which was estimated to be 10 500 cu. ft., and this is equivalent to a flow of 23.3 cu. ft. per sec. for 7.5 min. Thus, the total discharge of the two siphons was about $390 + 9.5 + 23.3 = 422.8$ cu. ft. per sec. The minimum cross-sectional area of the siphons is 8 sq. ft. each, or 16 sq. ft. for the two, hence, the velocity of flow was $\frac{422.8}{16} = 26\,425$ ft. per sec. As the theoretical velocity, under the 25.55-ft. head, is 40.54 ft. per sec., the efficiency of the siphons was $\frac{26.425}{40.54} = 0.652$. As four of the siphons act under a head of 18.5 ft. and four under a head of 26.5 ft., the total capacity of the spillway is:

$$Q = (0.652 \times 32)\sqrt{64.32 \times 18.5} + \sqrt{64.32 \times 26.5} = 1\,581 \text{ cu. ft. per sec.}$$

In this case, the velocity head is 10.85 ft., the entry head is 0.79 ft., and the friction head is 3.04 ft., leaving a head of 10.87 ft. to overcome the resistances at bends and of conversion in the upper leg of the siphon.

Another spillway, at Rochester, N. Y., was built in the forebay of Plant No. 2 of the Rochester Railway and Light Company, to discharge the water flowing in the head-race whenever a sudden shut-down of the plant occurs. This spillway consists of a single siphon, 3 ft. high by 16 ft. wide, which acts under a head of about 17.5 ft. The lower leg has a uniform cross-sectional area of 48 sq. ft., but changes in section from 3 by 16 ft. at the throat, to 4 by 12 ft. at the outlet, where the siphon connects with a large pipe that carries the water over a cliff into the Genesee River. A chute, for disposing of ice from the forebay, joins the pipe, just beyond the siphon outlet. This siphon requires from 10 sec. to 5 min. to get under way, depending on how rapidly the water rises, and the fluctuation of the water surface is from 1.5 to 2.0 ft. No accurate determination of the discharge has been made, but it has been observed that a sudden closing of the turbine gates, when the wheels are drawing 1 200 cu. ft. per sec., causes a rapid rise in the forebay water level followed by a quick drop, which indicates that the spillway capacity is considerably in excess of 1 200 cu. ft. per sec. From this, it appears that the siphon has a high efficiency, probably in excess of 75 per cent.

The siphon shown in Fig. 9 illustrates a low-head spillway for regulating the water level in an abandoned section of the Erie Canal, which is used as a storage reservoir by the General Electric Company at Schenectady, N. Y. This spillway consists of three siphons, each 1.75 ft. high by 4 ft. wide. At times of high water in the Mohawk River, the head may be reduced to 4 ft., which will give a discharge of about 200 cu. ft. per sec. The lower leg is inclined backward, so that the priming jet will strike high up on the far wall of the leg and the volume of air entrapped above the diaphragm will be a

minimum. The crests of the siphons are at the normal water level and the air vents are at varying elevations so that they will be sealed successively, when the water rises 0.7 ft., 0.8 ft., and 0.9 ft., respectively. Priming is practically instantaneous in each siphon, as soon as the air vent is sealed. Since this spillway was built, it has been found desirable to reduce the fluctuation of the water level. This has been accomplished by inserting in the air vents, metal tubes which extend downward to the desired level.

A siphon spillway is used in the Alpine Dam, in Lagunitas Creek Canyon, a short distance north of San Francisco, Calif. The dam is a concrete structure curved in plan, of gravity section, and about 100 ft. high, and impounds water for the supply of various municipalities of the Marin Water District. The spillway occupies a width of 64 ft., in the middle portion of the dam, and consists of six siphons, each 3 ft. high by 7.5 ft. wide. The capacity is

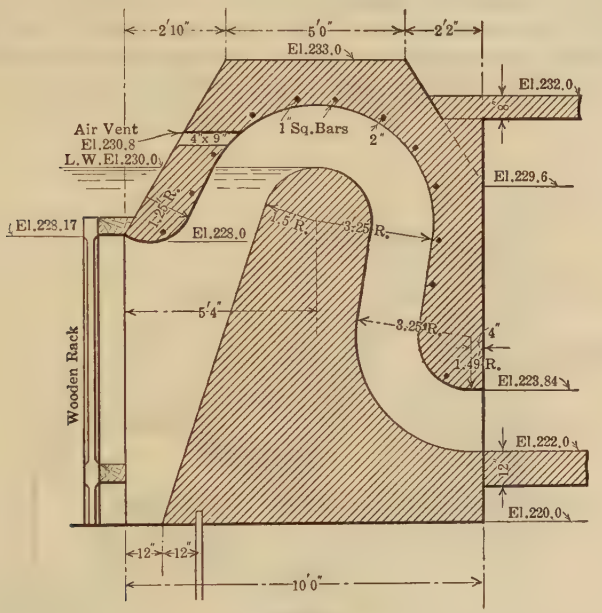


FIG. 9.

somewhat in excess of 4 000 cu. ft. per sec., under a head of 31 ft. The lower leg of the siphon is vertical and is of uniform section, terminating at a bend that deflects the water out to the face of the dam, which is formed on a curve of large radius, so that the water will not jump clear of the masonry. Parapet walls confine the outflowing water down the face of the dam, converging to a width of 35 ft. at the bottom, and the discharge is into the natural channel of the stream. The air vents in the various units are at slightly different elevations, which makes the siphons prime and cease flowing successively and thus vary the discharge of the spillway as the reservoir level changes, although a rise of 1 ft. will bring all the siphons into operation. The original plan for this dam contemplated a waste weir at one end, with a waste channel to carry the water. This would have involved heavy rock excavation,

so the siphon spillway was adopted. An overflow waste weir, with a length equal to the siphon spillway, would require a depth of about 6.5 ft. of water on its crest, in order to discharge 4 000 cu. ft. per sec., and to discharge this flow with a depth of 1 ft. on the crest would require a length of about 1 140 ft.

The Hetch Hetchy Dam, across the Tuolumne River, in California, now under construction to provide a water supply for the City of San Francisco, will have a siphon spillway consisting of eighteen siphons. All these siphons will be 4 ft. high, with fourteen units, 10 ft. wide, and four units, 9 ft. 2 in. wide. The inlet of each siphon will be 8 ft. below the water surface and will have an area double that of the lower leg. As the flow line of the reservoir to be created by the dam will be 3 620 ft. above sea level, the siphonic head will be 29.5 ft. Near the bottom of the lower leg in each siphon, there will be a reverse bend to deflect the flow of water across the conduit, so as to seal the leg. This will cause the siphons to prime when there is a rise of from 1.0 to 1.5 ft. in the reservoir level. The outflowing water, after leaving the siphons, will flow over steps in the face of the dam and drop into the river channel, 180 ft. below. The capacity of the spillway will be about 20 000 cu. ft. per sec.

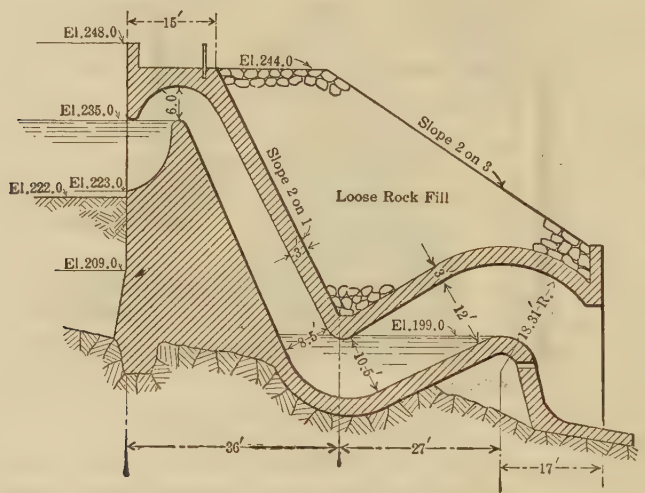


Fig. 10.

The Sweetwater Dam, near San Diego, Calif., has an overflow spillway at one end and a siphon spillway at the other. The siphons are shown in Fig. 10. There are six siphons, each 6 ft. high by 12 ft. wide, acting under a head of 33.5 ft. The inlets are 12 ft. high by 12 ft. wide and the outlets are 12 ft. high by 8.5 ft. wide. The lower legs which are inclined, are of practically uniform area throughout, varying from 6 by 12 ft. at the throat to 8.5 by 8.5 ft. at the bottom, and terminate in a sealing basin. The discharge capacity of this spillway is estimated to be about 12 000 cu. ft. per sec.

A spillway on the Arizona Canal of the Salt River Project of the U. S. Reclamation Service, near Phœnix, Ariz., consisting of several units, was

built to control the water level at a power plant. The siphons are provided with an air valve, at the top of the crown, to relieve pressure caused by a rise of the water level in the sealing basin, which would retard or prevent priming. This spillway acts under a head of 18.5 ft.

Another small siphon, built by the U. S. Reclamation Service, is at the head of the East Park Feed Canal of the Orlando Project, in California. This spillway consists of a single siphon, 1 ft. high by 6 ft. wide, which operates at a rise of 0.2 ft. in the water level and under a head of 17 ft., and has a capacity of about 100 cu. ft. per sec.

The Huntington Lake Siphon Spillway, on Big Creek, a tributary of the San Joaquin River, in California, about 45 miles northeast of Fresno, consists of seven siphons and was built to supplement the discharge over the main

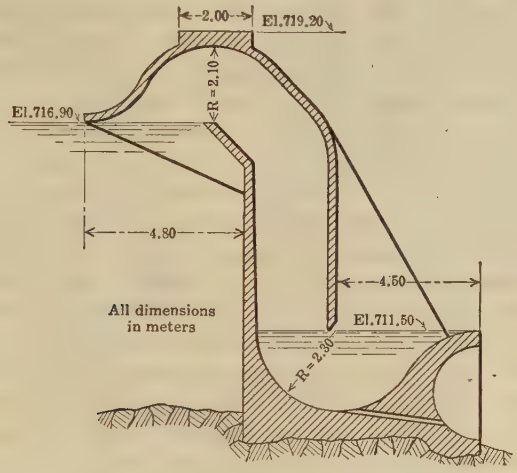


FIG. 11.

dam, which has a crest length of 645 ft. The throat of each siphon is 3.5 ft. high by 12 ft. wide. The inlets are 4 ft. below the normal water level and are 9.25 ft. high by 15.5 ft. wide. The upper legs converge in section to the throat area of 42 sq. ft. The lower legs are of uniform area throughout, but vary in length for the different units. In five siphons, the lower leg is inclined backward and in the other two siphons the leg slopes forward. Sealing basins are provided at the outlets, and each siphon has an 8 by 27-in. air vent, extending from the face of the spillway to the crown. These vents are sealed when the water rises above the crest of the main dam. Siphons Nos. 1, 2, and 3, operate under a head of 12 ft., Nos. 4 and 5, under a head of 15 ft., No. 6, under a head of 18 ft., and No. 7, under a head of 22 ft. The total capacity of the spillway is estimated to be about 5 500 cu. ft. per sec.

There are a number of siphon spillways on the canals of the East Contra Costa Irrigation District, near Brentwood, Calif., which operate under heads of from 17 to 22 ft. The siphons are 3 in. high, and the standard width of units is 6 ft. The spillways consist of several siphons of standard width and a narrower siphon, as may be required to give the desired capacity.

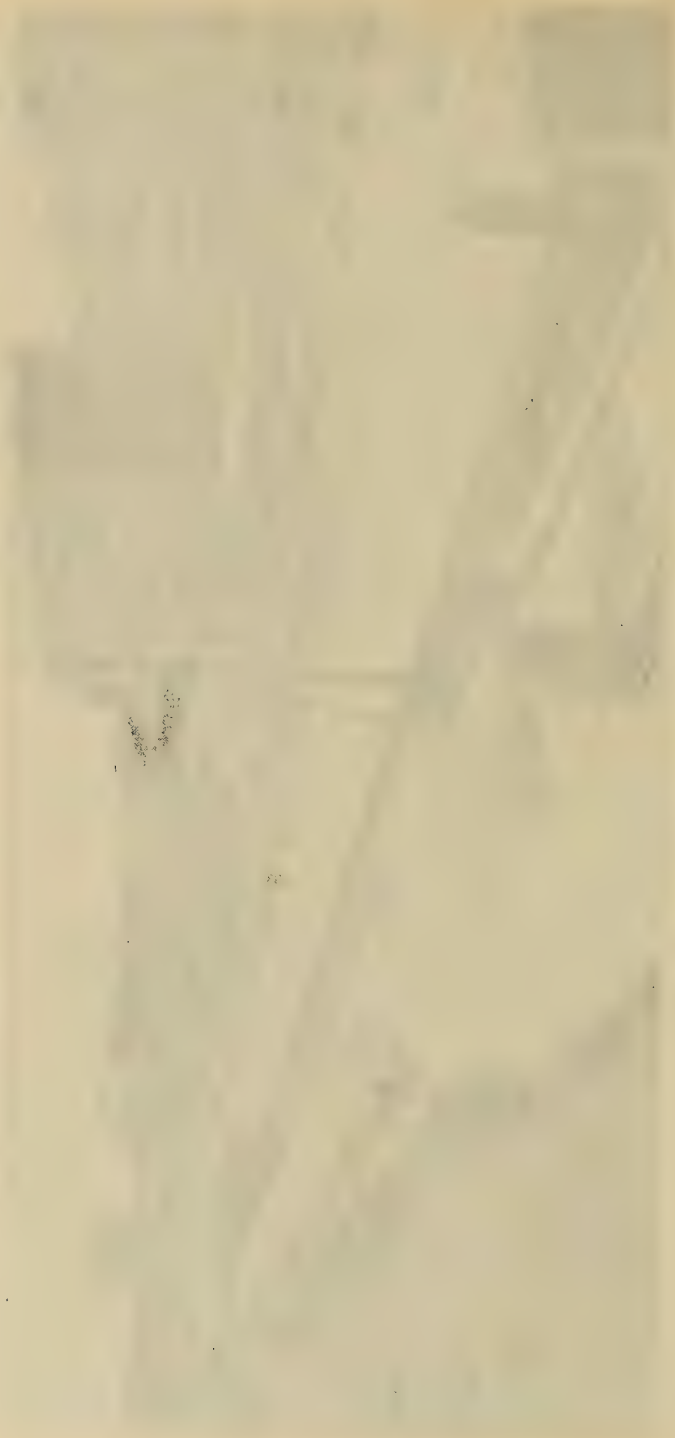
The siphon shown in Fig. 6 illustrates a spillway about 3 miles northeast of Ripon, Calif., at the end of the main distributing canal of the South San Joaquin Irrigation District. This spillway consists of two siphons, each 2 ft. high by 3 ft. wide, with a radial, steel flood-gate between the siphons. The main siphons are 0.5 ft. above the normal water level and below these are auxiliary priming siphons, 2 in. high by 3 ft. wide, immediately above the water level. The priming siphons discharge into sealing basins at the outlets of the main siphons. As the operation of this type of spillway has been described, no further explanation is necessary, except to state that a rise of 0.2 ft. in the water level seals the air vent of the main siphon and starts the priming siphon in operation, and that the time required to prime the main siphon is about 5 min. The capacity of the two siphons, under 8.25-ft. head, is about 200 cu. ft. per sec.

A siphon spillway at the Badana Dam, near Genoa, which is typical of a number of such structures in Italy, is shown in Fig. 11. This spillway consists of six siphons, each 6.9 ft. high by 6.2 ft. wide, and operates under a head of 17.7 ft., with capacity of 3 180 cu. ft. per sec.

A siphon with manually controlled operating device may be used in place of a sluice-gate, as in Lock No. 8 of the Oswego Canal, at Oswego, N. Y. The lock-chamber is filled and emptied through culverts in the side-walls bent up above the upper water surface, forming siphons as shown in Fig. 12. The siphons are put in operation by exhausting the air, entrapped in the conduits, into vacuum tanks, one of which is located in each lock wall, as shown in Fig. 13. These vacuum tanks are connected with the upper pool by 12-in. pipes, with gate-valves to regulate the flow, and are also connected with the lower pool by 20-in. pipes controlled by gate-valves. A connection between the top of the siphon and the top of the vacuum tank is made with a 4-in. pipe on which are two valves and a pressure-gauge. Valve *A* cuts off the flow in the pipe and Valve *B* serves to admit air to the siphon.

The operation is as follows: The 20-in. outlet valve is closed and the tank is filled with water from the upper pool through the 12-in. pipe; the valve on the 12-in. pipe is then closed and the valve on the 20-in. pipe is opened. The water does not run out of the tank, but is sustained by the atmospheric pressure on the water surface of the lower pool, in which the outlet pipe is submerged. On opening Valve *A* (Valve *B* being closed), the water rushes out of the tank and is replaced by air from the siphon. This air expands to fill the tank, losing density and pressure in the process, so that the atmospheric pressure, on the water surface of the upper pool, causes water to rise and flow through the siphon. This flow of water carries out the remaining air in the conduit, and the full siphonic flow is established. The flow of water through the crown of the siphon then exhausts what air remains in the tank, carrying it down into the culvert where it is carried along by the water. This air, being compressed by its plunge below the water surface, rises to the top of the culvert and is collected at a recess in the masonry, from which it escapes through a pipe that extends up through the wall to a point above the upper water level, and issues from the pipe with a loud sound. When the air is exhausted from the vacuum tank, it is replaced by water forced up

Fig. 12.—View of Siphon Lock, Oswego, N. Y.



through the 20-in. pipe by the atmospheric pressure on the surface of the lower pool. As soon as the tank is completely filled with water, Valve *A* is closed. The siphon continues in operation until the lock-chamber is filled or emptied,

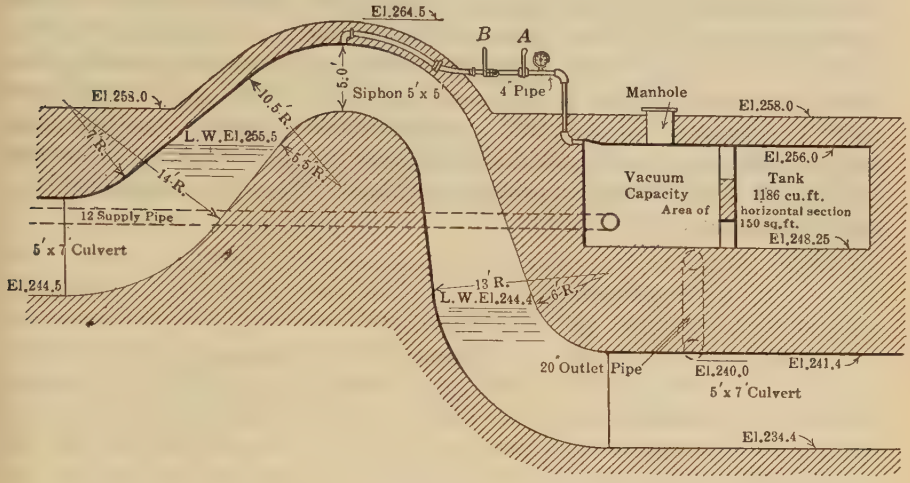


FIG. 13.

depending on whether the upper or the lower unit is being used, and when the flow ceases Valve *B* is opened to admit air to the siphon. The siphons are 5 by 5 ft. in cross-section at the throat, and the normal lift of the lock is 11.1 ft. The lock-chamber is filled in from 4.5 to 5 min. and is emptied in from 5.5 to 6 min., the difference being due to the location of the vacuum tanks at the head of the lock, near the upper siphons. With daily lockages, it is not necessary to fill the vacuum tank with water from the upper pool, except for the initial filling at the beginning of the season, the operation of the siphon, from time to time, being sufficient to keep the tank filled.

DISCUSSION

CHARLES F. RUFF,* JUN. AM. SOC. C. E. (by letter).—To the many useful and ingenious applications of the siphon, which Mr. Stickney describes in his interesting paper, the writer wishes to add one that he saw at a small hydro-electric plant in the French Pyrenees. This plant was equipped with four 3 500-h. p. Pelton wheels, operating under a head of 926 m., or more than 3 000 ft. The water was delivered from the forebay through a pipe line down the mountain side, and it was desired to have an automatic shut-off on this line in case of serious leak or break at the plant. This was accomplished by a siphon, arranged as shown in Fig. 14.

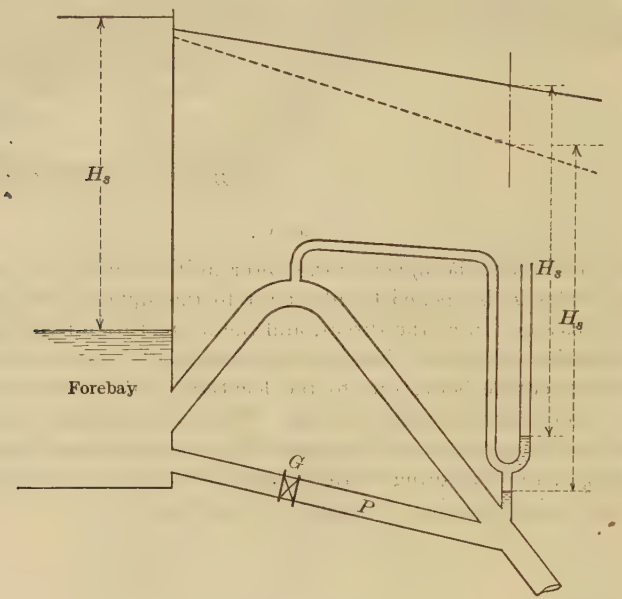


FIG. 14.

The siphon was started by allowing water to flow through the by-pass, P', until a good flow was established; the gate, G, was then closed and the weight of the water in the penstock caused sufficient vacuum to prime the siphon. Under normal conditions, the hydraulic gradient was somewhat as shown by the solid line, and the water in the free end of the U-tube stood a distance below it, equal to the siphonic head, H_s. When a break occurred, or, for any reason, the flow increased beyond a certain limit, the hydraulic gradient fell to the position shown by the dotted line, and the water in the U-tube dropped correspondingly, admitting air to the siphon and stopping the flow.

The writer does not know whether this siphon was ever called on in an emergency, nor what tests were made to determine its effectiveness.

* With Water Dept., City of Long Beach, Forest Hills, N. Y.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The author deserves much credit for an interesting paper on a subject about which little has hitherto been written. He is especially well qualified to discuss this subject, as it was largely through his efforts that siphon spillways were introduced, in 1909, on the New York State Barge Canal, and, afterward, on a number of dams and canals in various parts of the United States.

In Europe, siphon spillways had been built nearly forty years before the advantages of this type of spillway were appreciated in this country. The first siphon spillway, of which the speaker can find any record, was constructed in connection with a dam, at Mittersheim, in Lorraine, France. It consisted of a large vertical pipe, about 2.33 ft. in diameter, having a bend at its upper end. A 6-in. pipe, provided with an ejector, was connected to the bend. As soon as the water in the reservoir rose to its normal level, the ejector came into play and forced the air out of the bend of the large pipe, thus causing siphonic action to take place in the large pipe before the water surface in the reservoir had risen above the top of the bend of the siphon pipe. The use of a small ejector pipe, which was liable to become clogged by débris, was an objectionable feature of this arrangement, and the circular form of the siphon was uneconomical and only practical for siphons of small diameter.

An improvement in these respects was soon made by J. Heyn, of Stettin, Prussia, who was the first engineer to build a siphon spillway without an ejector. This siphon which was placed on the flood-gate of an irrigation canal in East Prussia, was made of riveted sheet iron, and was $1\frac{1}{2}$ by $10\frac{1}{2}$ ft. in cross-section. It operated under an available head of $4\frac{1}{4}$ ft., and showed an efficiency of about 50 per cent. The priming is effected by the flow of water over the lower surface of the crest of the siphon, which draws quickly, by friction, the air from the upper part of the siphon, and thus causes siphonic action. As soon as the water level is lowered below the air passages, provided at the bend of the siphon, at normal water level, air enters the pipe and siphonic action is broken. The water is drained from the curve at the outlet end through small holes, in order to prevent its being frozen in cold weather.

Almost contemporaneously with Heyn, Gregotti began the building of siphon spillways without ejectors, in Italy, constructing them of reinforced concrete. One of the largest of these is on the Canale di Milano, near Verona. It consists of ten units, each of which has a cross-sectional area of 14 sq. ft. and an available head of 20 ft. These siphons have an efficiency of 41 per cent. Still larger siphon spillways have been built in France, and a number have been constructed in Switzerland, Sweden, and other countries.

The siphon at Gibswil, Switzerland, consists of a $\frac{1}{4}$-in. riveted steel pipe, which tapers from a diameter of 31.5 in. at the upper end to 23.6 in. at the lower end, in order to prevent the column of water from parting under a head of 52.48 ft. Of course, the effective head cannot exceed that due to a vacuum, which is 33.9 ft. at sea level. At the normal surface, the pipe is cut on a horizontal plane and covered by a reinforced concrete hood, which projects 1 m. below the normal water level to prevent the siphon from being clogged

* Cons. Engr., New York City.

by ice in winter. The air holes are narrow slots in the hood at the normal water level of the reservoir.

Mr. Stickney has stated that the first siphon spillways in the United States were built, about 1909, on the New York State Barge Canal, according to his designs. In 1896, however, the late J. James R. Croes, Past-President, Am. Soc. C. E., showed the speaker plans he had developed for a small dam which was to be constructed at Princeton, N. J., to form a reservoir now known as Carnegie Lake. This dam which was built in 1905, consists of piers placed 28 ft. apart, and joined by curtain walls, 4 ft. thick, in which I-beams were placed. The original plans contemplated the use of a siphonic spillway, but owing to the opposition of the Pennsylvania Railroad Company, the interests of which were dominant, Mr. Croes reluctantly abandoned the idea of building a dam with a siphon spillway and submitted other plans.

TABLE 5.—Siphon Spillways, Yuma Project.

Date.	Siphon number.	Head, in feet.	Theoretical velocity, in feet per second.	Discharge, in Cubic Feet per Second.		Coefficient.
				Theoretical.	Measured.	
July 24, 1912....	4, 5	11.61	27.3	620	484.9	78.3
24, "	4, 5	11.80	27.5	625	479.6	77.0
25, "	3, 4, 5	11.77	27.4	933	693.5	74.3
25, "	3, 4	11.90	27.7	630	443.3	70.2
26, "	3, 5	11.85	27.6	627	505.1	80.5
26, "	3, 4, 5	11.60	27.3	980	668.0	71.5
29, "	A 11	10.65	26.2	1 488	1 043.4	70.4
29, "	1, 2, 4, 5	11.00	26.6	1 205	853.1	71.0
29, "	A 11	10.87	26.4	1 498	1 012.7	68.0
August 5, "	3	11.90	27.7	315	243.3	77.5
7, "	1, 2	11.60	27.3	620	405.0	65.5
7, "	1, 2	11.60	27.3	620	415.0	67.1
8, "	2, 3	11.48	27.2	617	419.3	68.0
8, "	2	11.87	27.6	314	208.0	66.5
19, "	1, 2	10.98	26.7	606	432.5	71.5
19, "	2	11.82	27.6	314	202.0	64.4

TABLE 6.—Observed Depth of Water Over Lip to Start Siphon.*

Tube number.	Outlet submerged 5.35 ft.	Outlet submerged 6.0 ft.
1	0.40	0.55
2	0.35	0.45
3	0.15	0.25
4	0.35	0.45
5	0.40	0.50

* After a number of leaks were closed in Tubes Nos. 1 and 2, they primed at depths about 0.10 ft. less than those shown in Table 6. The seal broke in a few minutes when the water surface had dropped about 0.1 ft. below the air valve.

In 1916, the speaker was consulted by the late W. W. Scranton, President of the Water-Works of Scranton, Pa., about the stability of five dams. These dams were designed by an eminent engineer, the late Alphonse Fteley, Past-President, Am. Soc. C. E. To increase the storage in the reservoirs, the crests of the dams had been raised 3 ft. which caused the line of pressure, when the reservoir was full, to fall a trifle outside of the center third of the profile. This eccentricity was less than that occurring in the profile type designed

by Rankine, who was the first engineer to call attention to the necessity of designing a dam so that the lines of pressure would be within the center third of the profile, in order to avoid tension in the masonry. However, the engineers of the Water Supply Commission of Pennsylvania insisted that the dams be cut down to their original heights. This might easily have been obviated by constructing a siphonic spillway designed so as to maintain the water at its original normal level. The speaker, with the assistance of Mr. A. G. Hillberg, designed a siphon spillway for the largest of the Scranton dams. It was to be built, on an inclined plane, in the outer slope of an earth dam, like the Gibswil Dam in Switzerland, but was to be constructed of reinforced concrete instead of metal. The siphon was to consist of five units, each 4 by 6 ft. in cross-section, and was to have a capacity of about 3 600 cu. ft. per sec. This siphon spillway which could have been constructed at a moderate expense, would have been at that time the largest siphon spillway in America. Several siphon spillways had already been built in the United States, all of which were operating successfully. For reasons unknown to the speaker, probably as a matter of policy, Mr. Scranton decided to reduce the heights of these dams to their original levels. It proved to be quite an expensive operation, as the dams were built of excellent masonry.

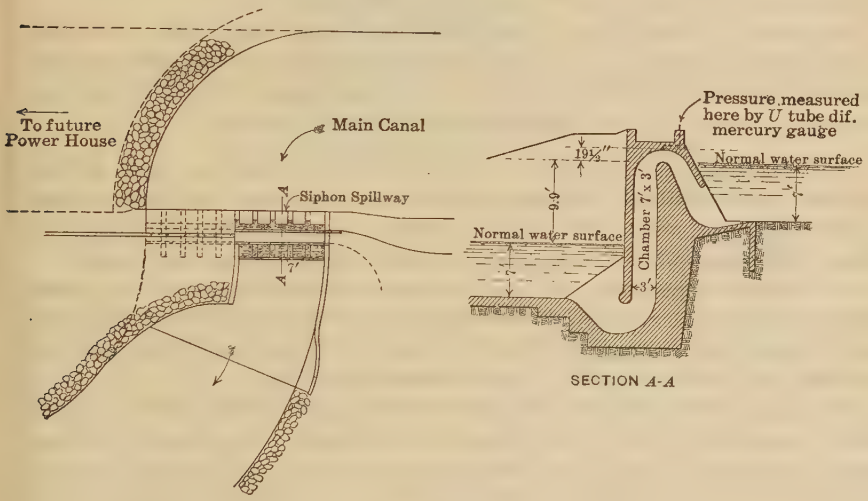


FIG. 15.

In the Alpine Dam, in California, Mr. Stickney has given an excellent example of the economy that may be effected by the use of siphon spillways. This dam which is 100 ft. in height, is equipped with a siphon spillway consisting of six units, each 3 by 7.5 ft. in cross-section. This spillway occupies a crest width of 64 ft. in the central part of the dam, and, according to the author, discharges as much water, with a head of 1 ft., as an ordinary overflow weir, 1 140 ft. long, would discharge with the same head. Compare this dam with the famous New Croton Dam, built, 1895 to 1907, to obtain a large storage reservoir for New York City. It forms a masonry wall across

the valley, the southern part being the main dam, and the northern part the overflow weir which is built along the rocky side-hill on the north side of the valley, nearly at right angles to the main dam. The main dam and the waste weir are 1 200 ft. and 1 000 ft. long, respectively. The waste channel, excavated in rock along the north side of the valley, is 50 ft. wide at its upper end and 125 ft. wide at its lower end, where it is spanned by a 200-ft. steel bridge, the superstructure of which cost $40 000. The speaker has no doubt that a considerable saving might have been made if this dam had been built straight across the valley and provided with a siphon spillway at its northern end.

A number of comparatively small siphon spillways have been built by the U. S. Reclamation Service, and interesting details relative to their construction have been given* by A. T. Mitchelson, Senior Irrigation Engineer. A number of tests made on small models of siphon spillways at the Throop College of Technology in California and at Fort Collins, Colo., are mentioned by Mr. Mitchelson.

The efficiency of some of the siphon spillways built by the U. S. Reclamation Service has been determined. The data for Tables 5 and 6 which give the results of some of these tests, were furnished the speaker by the Director of the Service, A. P. Davis, Past-President, Am. Soc. C. E.

The siphon spillway, Fig. 15, on which these tests were made, is on the Yuma Project in Arizona. The first siphon spillways had efficiencies of only about 50%, but with modern siphons, this factor, as shown by Table 5, has been increased to from 64 to 80% by improvements in construction.

W. P. Creager,† M. Am. Soc. C. E.—The author's excellent paper will be welcomed by all engineers interested in the economical control of the discharge of water from reservoirs. The speaker wishes to discuss several advantages of siphon spillways to which the author, he thinks, has not given full credit.

Mr. Stickney states that the head of water corresponding to the atmospheric pressure is the greatest head that may be utilized for producing siphonic flow. The speaker will endeavor to prove that a head considerably greater than the "siphonic limit" of 33.9 ft. may be used successfully.

For a pipe discharging full, the slope of the hydraulic grade, at any point, varies approximately as the total head on the pipe. The pipe will always remain full if it is tight and the hydraulic grade line, as used in Fig. 1, is above the top of the pipe at all points. If these two laws can be accepted as correct, the siphon indicated in Fig. 1 can be reproduced as shown in Fig. 16, acting under two heads, one of which (like that given by the author) is less and the other greater than the "siphonic limit" of 33.9 ft. The elevation of the hydraulic grade at the entrance to the siphon is the same for the two cases, but the hydraulic grade at the outlet of the longer siphon is below that of the shorter siphon a distance equal to the difference between the total heads.

As the slope of the hydraulic grade of the long siphon is uniformly greater than that of the shorter siphon, the discharge of the long siphon is the greater

* "Spillways for Reservoirs and Canals", *Bulletin No. 831*, U. S. Dept. of Agriculture.
† Engr. of Hydr. Structures, The J. G. White Eng. Corporation, New York City.

if the pipe remains full, and it will remain full if the hydraulic grade is above the top of the pipe. The siphonic limit is reached only when the lower leg is prolonged to such an extent that the hydraulic grade falls below the top of the pipe.

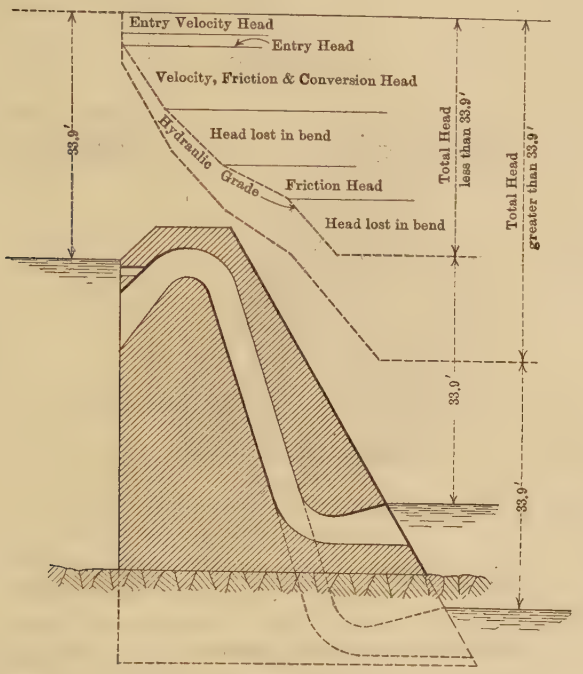


FIG. 16.

As an example indicating the application of the principles governing the design of siphons, the following is given: In Fig. 17, the head acting on the siphon is 36.0 ft. The shape and area of the siphon are constant from *B* to *E*. The friction loss for a straight pipe is approximately:

$$h_f = k\, l\, v^2$$

For the constant section of this siphon it has been determined from accepted formulas that:

$$h_f = \frac{l\, v^2}{12\,000}$$

The velocity head is,

$$h_v = \frac{v^2}{2\, g}$$

The loss at the bends is calculated, according to experiments, in percentages of the velocity head, or,

$$h_b = \frac{p\, v^2}{2\, g}$$

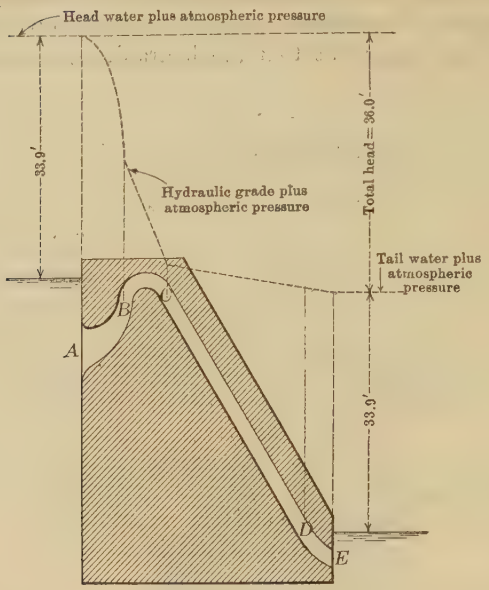


FIG. 17.

From A to B:

Velocity head $\dfrac{v^2}{2g} = \dfrac{v^2}{64.4}$

Friction head probably about $\dfrac{v^2}{6\ 000}$

Head lost at bend $\dfrac{1}{8} \cdot \dfrac{v^2}{2g} = \dfrac{v^2}{515}$

From B to C:

Friction head $\dfrac{8\ v^2}{12\ 000} = \dfrac{v^2}{1\ 500}$

Head lost at bend $\dfrac{0.9\ v^2}{2g} = \dfrac{v^2}{71.6}$

From C to D:

Friction loss $\dfrac{40\ v^2}{12\ 000} = \dfrac{v^2}{300}$

From D to E:

Friction loss $\dfrac{5\ v^2}{12\ 000} = \dfrac{v^2}{2\ 400}$

Head lost at bend $\dfrac{0.2\ v^2}{2g} = \dfrac{v^2}{322}$

The summation of these values is:

$$\frac{36.3\ v^2}{1\ 000} = \text{total head} = 36.0$$

Therefore,

$$v = 31.50 \text{ ft. per sec.}$$

The theoretical discharge velocity, without friction, is:

$$v' = \sqrt{2\,g\,h} = 8.03\,\sqrt{36} = 48.18$$

The coefficient of discharge, therefore, is:

$$c = \frac{31.50}{48.18} = 0.654$$

Using the calculated velocity of 31.50, the fall of the hydraulic grade is calculated in Table 7 and the grade is plotted in Fig. 17.

TABLE 7.

Section.	Velocity head, in feet.	Friction head, in feet.	Head lost at bend, in feet.	Total head, in feet.
A to *B*.....	15.39	0.16	1.92	17.47
B to *C*.....	.....	0.66	13.83	14.49
C to *D*.....	.....	3.30	.....	3.30
D to *E*.....	.....	0.42	0.32	0.74
.....	.....	.....	.....	36.00

The hydraulic grade, for this case, is seen to be entirely above the siphon and the head is not too great for proper operation.

The Ocoee Siphon, indicated in Fig. 8, designed and tested under the speaker's direction, is believed to be the first siphon constructed in the United States, the successful operation of which depended on the time of priming.

The time required to seal and prime the Ocoee Siphon, as indicated by the author in Table 4, is not strictly correct on account of his misinterpretation of the speaker's report of the tests. The time given in Table 4 indicates the total interval between the cessation of flow at the forebay outlet and the priming of the nearest siphon. As a perceptible interval of time was required for the wave, incident to the stopping of the flow, to pass from the forebay outlet to the siphon, the total time of priming should be corrected accordingly. The correct interval between the start of the rise of the water surface at the siphon, the moment of priming, and the revised calculations necessary to compute the time of priming, according to the author's method, are given in Table 8.

TABLE 8.

Test number.	Elevations of Water, in Feet:		Rise, in feet.	Corrected time, in seconds.	Rate of rise, in feet per second.	Time Required, in Seconds:	
	Low.	High.				To seal.	To prime.
2	1 089.20	1 089.55	0.35	19	0.01843	13.58	5.42
4	"	1 089.57	0.37	14	0.0264	9.47	4.53
5	"	1 089.75	0.55	9	0.0611	4.09	4.91
6	"	1 089.95	0.75	5	0.1500	1.67	3.33

It will be noted that, contrary to the author's deductions, the time required to prime is not constant, but tends to decrease as the rate of rise increases. This seems logical when it is considered that the time required to prime is that required to remove the air trapped in the throat of the siphon. This time is proportional to the average flow during the operation of priming and this average flow increases with the rate of rise of water surface.

The conditions under which the Ocoee tests were made, were such that great accuracy was not possible. The results of these tests, therefore, cannot be used without the application of a considerable margin of safety. Fig. 18 shows the approximate relation between the rate of rise of water surface and the time required to prime, as indicated by the four Ocoee tests.

The curved line drawn through these points is intended to indicate the time of priming of these siphons as nearly as it is possible to determine it from the tests. If the water surface was made to rise suddenly to an elevation considerably above the top of the siphons, it is believed that the time required to prime would correspond approximately to the time required for the water to fall a distance equal to the length of the lower leg, or about 1.25 sec. for a 25-ft. siphon. Therefore, the curve is intended to be asymptotic to a speed of 1.25.

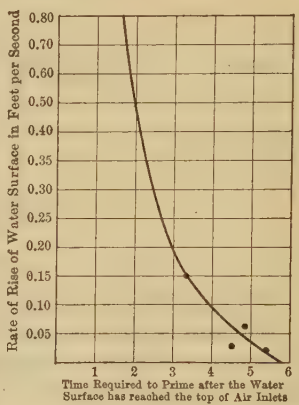


FIG. 18.

In this type of siphon, if the water surface was raised to an elevation just above the top of the air inlets and held stationary, the curve indicates that the siphons would probably prime in about 5.8 sec.

Suppose it is determined from the area of the forebay and the rate of discharge to the turbines that the rate of rise of water surface for full closure would be 0.5 ft. per sec., then Fig. 18 indicates that the probable time required for the siphons to prime would be 2 sec. Hence, the probable maximum rise of water surface above the top of the air inlets would be $2 \times 0.5 = 1.0$ ft.

However, until such time as more accurate experiments have been made to supplement the speaker's curve, it is believed that at least twice the calculated maximum rise should be used.

Siphon spillways are reliable if designed properly and built where they are not exposed to the danger of being obstructed by trash or ice. They have been used frequently in exposed locations where a failure to operate merely results in inconvenience or the loss of a few flash-boards and where they are not relied on to discharge part of the flood flow. In such cases, trash racks may be used to prevent the entrance of logs, trees, or ice, which might clog the siphons, and their failure to operate would be only the result of neglect.

They are suitable for the passage of flood flows only at the outlet of large reservoirs where no submerged trash of sufficient size to clog them will reach the dam and where an auxiliary open spillway is provided to pass floating débris and ice. For flood discharge, the siphons should not be provided with trash racks and the throat area should be as large as possible.

The air inlets need constant inspection in cold weather and when trash is running. The speaker knows of one instance where the air inlets became clogged with ice and the siphons drew the water out of the pond and caused the shut-down of the hydraulic turbines. Such instances are rare and may be remedied by an attendant, in a few seconds.

H. F. Dunham,* M. Am. Soc. C. E.—Doubtless many engineers will be surprised to learn that so large a number of siphons, of so many varying designs, are in existence. The objects to be gained by their use also are different.

The Taintor gate, it would seem, has certain advantages as a flood-gate. When wide open, the flow is direct, with little opportunity for obstruction. In river channels, the siphon, because of its more intricate passages, might become choked with ice, fences, tree trunks, fragments of buildings, and the like, or, in Southern rivers, with plant-rooted soil which rises in great quantities and floats, particularly at flood periods. Information about the range of comparative usefulness of the siphon with respect to ice and climate would be appreciated.

If a siphon is in masonry and not metal-lined, small overflows, such as from waves, and especially small leaks through the masonry, would be likely to be frozen in cold weather and thus obstruct or close the opening. The reference to absence of frost in a sewer manhole is not comparable, for the sewer is deep in the earth, contains gases and decomposing substances, and is not subject to cold-air currents. In some of the siphons shown by the author, the discharge opening is exposed and, in low-water periods, the siphon would act as a chimney through which currents of cold air would rush with up-stream cold winds. Ice from leakage has formed in great masses on the down-stream face of large dams and also in mine shafts, at depths of 600 ft., where the rock temperature was above 60° Fahr., caused by the cold air drawn down in ventilation.

Thus, it seems reasonable to inquire how the siphons operate under different conditions, how much attention they need, and whether, for small units, they have advantages over good regulating gates at places where men who have time to attend them, are always on duty? Data on the cost of construction of siphons as compared with cost of gates of an older, if not a simpler, type would be valued by the speaker.

Karl R. Kennison,† M. Am. Soc. C. E. (by letter).—The author has compiled a most valuable collection of data on the siphon spillway. In some of the installations which he has described, the requirements were ideally met by spillways of the siphon type. This is especially true of the various locks and canals. Under certain conditions, the siphon spillway is a satisfactory device for controlling the water level of a reservoir or a canal, but it has its limitations and there are some spillway requirements which it cannot meet.

Among the important requirements which the siphon can meet successfully are: A maximum discharge not to exceed a fairly well determined limit; small fluctuation of reservoir level to be confined to well determined limits in ordinary operation; spillway dam crest to be available for a roadway

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† Cons. Engr., Boston, Mass.

or other level working area; and available spillway length limited, requiring a high rate of discharge per unit of length.

Some of the important requirements which the siphon cannot meet with safety and economy are: Discharge of flood run-off of uncertain magnitude; provision for the extreme flood, say, once in fifty years, without material rise in reservoir level; certainty of operation in freezing temperature; no possibility of clogging with trash; and remote or inaccessible location with no dependence on periodical inspection.

The author described some installations that are more or less isolated and in locations where freezing temperatures are common; and it would be interesting to know the frequency of inspection required or what measures have been found necessary to insure proper maintenance and operation during the winter.

Ordinarily, the requirements of a spillway to prevent excessive head on, or overtopping of, an important dam cannot be met by the siphon type. There are exceptions, as in the case of the Hetch Hetchy Dam, built to part height, as described by the author. At this dam, it was desired to prevent overflow on account of the roadway. Furthermore, no suitable spillway site can be found at the sides of the steep gorge, until the dam is built to its full height. However, the important point in this case is that it is not essential to prevent overtopping the roadway. If an unusual flood or a clogging of the siphons should cause the water to flow over the top, it would result in only minor damage to the roadway. The concrete arched dam with its gravity section would suffer no damage.

All earth dams and most concrete non-overflow dams are not designed to withstand overtopping in unusual floods. They rely on the spillway to prevent it. Most of the spillways designed by the writer have been primarily safety valves. In each case, the degree of safety called for has varied greatly with the circumstances, but, in most instances, greater safety has been demanded than could be furnished by the siphon type of spillway.

The writer has used the siphon spillway in places where its proper operation in every emergency was not essential. In one instance, the requirements were unusual and were successfully met by this type. It was necessary to divert from the reservoir, automatically and without the use of submerged gates, which might leak, a well determined maximum rate of flow fixed by the capacity of the diversion conduit. It was also required to make this diversion at a certain fixed reservoir level and with the least possible fluctuation in this level. As the siphon allows a high rate of flow per unit of length, it was possible to enclose the entire device in a covered screen chamber, conveniently arranged for periodical inspection in all seasons, and thus exclude floating trash and experience little or no trouble from ice. Failure to operate at any time would result in some inconvenience, but no damage to life or property. It is in such situations that the siphon performs at its best, primarily for convenience and not for safety.

Joel D. Justin,* M. Am. Soc. C. E. (by letter).—The advantages of siphon spillways are manifest; the discharge per foot of crest is greater than

* Departmental Chf. Engr., The Ludlow Engineers, Inc., Winston-Salem, N. C.

with an overflow spillway and there is no operating machinery requiring expensive maintenance. It would also seem that, under ice conditions, there would be a great advantage, as the inlet could be located well below minimum head-water level and the outlet below tail-water level.

The author does not state whether it is the usual practice to protect the intake with trash racks. It would be interesting to learn to what extent trash and logs cause trouble in the operation of such siphons, and also to learn what rise in head-water elevation is necessary to prime the largest siphons, such as those at the Sweetwater Dam, shown in Fig. 10.

From Mr. Stickney's description of installations, it appears that it is now practicable to design and build siphons to pass water at the rate of about 120 cu. ft. per sec. per lin. ft. of dam, making due allowance for the necessary walls between the siphons. This places the siphon spillway in a position where it can compete, in the matter of capacity, in many cases with spillways equipped with Taintor gates. Large Taintor gates, however, have a greater capacity than this, and a spillway equipped with 20-ft. Stoney gates will discharge about 220 cu. ft. per sec. per lin. ft. of dam, and one equipped with 30-ft. Stoney gates will discharge about 400 cu. ft. per sec. per lin. ft. of dam, without any rise in head-water elevation.

However, every device has its limitations, and in cases where it is economical or necessary to pass the largest possible quantity of water over the shortest possible length of crest, some form of spillway gates, doubtless, will continue to be the correct solution of the problem.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—The design of siphon spillways, or any other hydraulic structure in which the principle of the water barometer is utilized, is not as simple as a casual reading of this paper might lead one to believe.

A point often overlooked in the design of these structures, is the possible effects of dissolved air on their operation. Water that has a free surface in contact with the atmosphere always contains, in solution, a definite proportion of air. The proportion, by weight, varies with the pressure and the temperature, but is constant for the same conditions. A diminution of pressure means the instant passage of a definite quantity of air from solution to suspension. The phenomenon is much more striking if the gas is carbon dioxide, as may be observed in a soda-water siphon.

In a true siphon, the water above the inlet contains, in solution, from 1½ to 3%, by volume, of air at atmospheric pressure. This is a true solution, that is, the dissolved air is in liquid form. As the water is accelerated into the inlet, the pressure is reduced to supply the acceleration head and, after passing the inlet, is further reduced by elevation and possibly, also, by acceleration. Air, therefore, is released from solution as the water approaches the crown. As the water is ascending, some of the air may escape from suspension and accumulate at the crown. If some of the atmospheric pressure has been used to create a high throat velocity, the absolute pressure at the crown may be low and the consequent release of air from solution may be great.

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At the crown and for some distance on either side, the motion of the water is sensibly horizontal, which facilitates the rising of the bubbles and their release from suspension.

While the water is passing down the discharge leg, the absolute pressure again increases, due to loss of elevation and possibly, also, to negative acceleration; but it may still be less than the saturation pressure for the quantity of air it carries, for a large part of the distance. The air bubbles will tend to rise against the velocity with the chance of further escape and accumulation at the crown. There is, therefore, a possibility of air accumulating at the crown in sufficient volume to stop the action of the siphon unless means are provided for its removal.

In order to be on the side of safety, the hydraulic grade plane should always be calculated through a proposed siphon, making liberal allowances for losses from friction and curvature. From these calculations, the absolute pressure at any point may be determined and the quantity of air released may be estimated. If the investigations show the possibility of "air-binding", an hydraulic ejector or some similar device should be provided to remove the air as it accumulates.

The working head on a siphon is not limited to the barometric height unless the tube is of uniform area throughout. By suitably contracting the discharge leg so that the water is continuously accelerated, it is possible to design for differences in level up to 50 ft., or more, and have the tube discharge full bore. The possibility of trouble from air, however, is much increased, and for high working heads the ejector may require more water than the siphon.

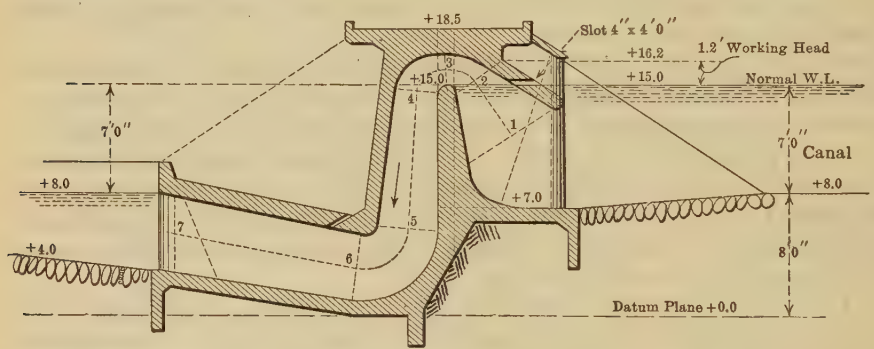

FIG. 19.

The discharge formula given by the author, is useful in fixing tentative dimensions to which more detailed calculations may be applied, but it should not be used to fix final dimensions, unless the tube is of uniform bore throughout; and, even then, the coefficient is a matter of much doubt owing to the losses from curvature and friction. If the cross-sectional area of the tube decreases and increases gradually, the coefficients may be more than unity and even approach 2.0. If the author's formula is applied to the crown area of the siphon shown in Fig. 19, it will be seen that, in this case, the coefficient is 1.7 by calculation and possibly more in reality. Any one having such

structures to design will be repaid in reading the chapter on siphons in "The Control of Water", by the late Philip à Morley Parker, M. Am. Soc. C. E., in which one will find that there are pitfalls which may trap the unwary.

Fig. 19 shows a section of a siphon spillway, designed for the main canal of the Lethbridge Northern Irrigation Project, Alberta, Canada. This canal heads in a stream that is subject to rapid fluctuations in discharge. About 3 miles below the intake is a steel flume with a limited free-board, and the siphon was designed to divert any surplus that might enter the canal and overflow the flume. The flume, designed for 800 cu. ft. per sec., might, in emergencies, carry an excess of 150 cu. ft. per sec. The possible fluctuation in 12 hours, in the quantity of water entering the canal, might amount to as much as 450 cu. ft. per sec. The spillway was accordingly designed to discharge 300 cu. ft. per sec., under a working rise of 1.2 ft., which corresponds to an excess of 450 cu. ft. per sec. in the canal.

Table 9 shows the calculations for this structure, and is self-explanatory. It will be noted (Line 22) that, at the crown, the water is under a negative pressure of 24.4 ft. of water (center of stream) or 25.4 ft. at the summit;

TABLE 9.—CALCULATIONS FOR SIPHON SPILLWAY.

		1	2	3	4	5	6	7
1	d = Depth, in feet	6.4	2.8	2.0	2.8	4.0	4.5	5.0
2	b = Width, in feet	4.0	4.0	4.0	4.0	4.0	4.0	8.0
3	$A = bd$ = Area, in square feet	25.6	11.2	8.0	11.2	16.0	20.0	40.0
4	$V = \frac{Q}{A}$ = Velocity, in feet per second	11.7	26.5	37.5	26.7	18.7	15.0	7.5
5	$h_v = \frac{v^2}{2g}$ = Velocity head, in feet	2.13	10.9	21.8	11.1	5.42	3.5	0.90
6	w = Perimeter, in feet	20.8	13.6	12.0	13.6	16.0	17.0	26.0
7	$r = \frac{A}{P}$ = Hydraulic radius	1.23	0.82	0.67	0.82	1.00	1.17	1.54
8	A_1 (mean)	....	18.7	9.6	9.6	13.6	18.0	30.0
9	r_1 (mean)	....	1.03	0.75	0.75	0.91	1.09	1.35
10	V_1 (mean)	....	16.3	31.2	31.2	22.0	16.6	10.0
11	$S = \left(\frac{V}{100\, r^{\frac{2}{3}}}\right)^2$, n (Kutter) = 0.015	....	.026	0.143	0.143	0.055	0.024	0.007
12	L = length, in feet	....	4.4	2.0	2.5	8.8	4.7	12.7
13	$hf = S\,L$ = friction loss, in feet	....	0.114	0.287	0.358	0.483	0.113	0.084
14	hc = loss from curvature, in feet*	....	....	2.39	2.39	....	0.98	....
15	δh	2.1	8.8	10.9	−10.7	− 5.7	− 1.9	− 2.6
16	H = head lost in section = $hf + hc + \delta h_v$	2.1	8.9	13.6	− 7.9	− 5.2	− 0.8	− 2.5
17	ΣH = Total head lost, in feet	2.1	11.0	24.6	16.7	11.5	10.7	8.2
18	$\Sigma hf + c$, in feet	0.0	0.1	2.8	5.6	6.1	7.2	7.3
19	Z = distance above datum, in feet	11.6	15.3	16.0	14.6	5.6	3.2	5.5
20	h_v	2.1	10.9	21.8	11.1	5.4	3.5	0.9
21	$h_v + Z + \Sigma hf + c = S$	13.7	26.3	40.6	31.3	17.1	13.9	13.7
22	P = Internal pressure, in feet = 16.2 − S	+ 2.5	−10.1	−24.4	−15.1	− 0.9	+ 2.3	+ 2.5

* The values of hc are as follows:

For Section 2-3, the mean value of $h_c = \frac{10.9 + 21.8}{2} = \frac{32.7}{2} = 16.35$ ft.

For Section 3-4, " " " " " " $= \frac{21.8 + 11.1}{2} = \frac{32.9}{2} = 16.45$ ft.

Mean, for Section 2-4 = 16.40 ft.

Total loss = $0.293\, h_v = 4.78$ ft.

For Section 5-6, the mean value of $h_v = \frac{5.40 + 3.50}{2} = \frac{8.9}{2} = 4.45$ ft.

Total loss = $0.22\, h_v = 0.98$ ft.

that is, with the water barometer at 29.5 ft., the absolute pressure is only about 4.1 ft. of water. Under the circumstances, and in spite of high crown velocity, air might accumulate at the crown. A hydraulic ejector is provided, therefore, to remove it. This ejector, under a pressure (absolute) of 4 ft. of water, will remove about 10 cu. ft. of air per min., which is more than the probable accumulation.

CHARLES G. DARRACH,* M. AM. SOC. C. E. (by letter).—An article in *Engineering News,* of March 14th, 1885, on a discussion by the writer of a paper read before the Engineers Club of Philadelphia on February 7th, 1885, includes a description of an automatic siphon weir at Mittersheim, which was designed to discharge large volumes with minimum change of level in the reservoir. This siphon weir which was invented by M. Hirsch, regulated the surface level of the reservoir within 0.03 m. MM. Belard and Hirsch planned and directed the work from 1864 to 1866, which was completed and in operation the following year.

A full description of this siphon weir may be found under the title, "Dam and Syphon Weir of the Reservoir at Mittersheim", in the notices on the models, charts, and drawings relating to the works of the "Ponts et Chaussées" and the Department of Mines, collected by order of the Ministry of Public Works and exhibited in the French collection at the Universal Exposition at Philadelphia, Pa., in 1876.

FRED A. NOETZLI,† ASSOC. M. AM. SOC. C. E. (by letter).—The author deserves much credit for having presented to the Profession such a comprehensive paper on siphon spillways. The various examples of existing structures of this kind, and the clear description of their action and efficiency, will undoubtedly prepare the way for a more general use of siphon spillways wherever the conditions are favorable.

The writer was particularly impressed by the various ingenious methods devised by the author for priming the siphon and sealing the lower leg against the entrance of air from below, without the use of a submerged outlet.

One of the main considerations entering the design of a siphon spillway is the determination of its efficiency. The author states that "properly designed siphons will have efficiencies of from 60 to 80%, but, for preliminary computations, the former figure should be used." The writer fails to find from the many examples cited by Mr. Stickney any definite information with regard to the best shape of siphon for which an efficiency of 75 to 80% might be effected. The tests cited gave efficiencies of about 60 to 65%, if one discards the figure derived for the spillway of Power Plant No. 2 at Rochester, N. Y., for which the data given are too uncertain to lead to a definite conclusion.

It would appear, therefore, that, at present, a factor of ignorance of from 20 to 30% (of 60%) is still involved in calculating the discharge of a siphon spillway. This seems to be rather high for a comparatively simple problem such as a siphon. There is no doubt that from a few systematic tests on siphons of different shapes, more definite conclusions might be drawn as to the shape of

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entrance and exit openings, radius of bends, etc., to obtain for siphons the maximum efficiency possible under conditions such as occur in practice. It is hoped that laboratory tests will be made on models and the results compared with accurate and systematic measurements for existing siphon spillways, in order to enable future structures of this kind to be built more economically than is possible with the present assumptions of "60 to 80%" efficiency.

For siphons of the type illustrated by Fig. 5, and used, for instance, for the Hetch Hetchy Dam, the water issues from the lower leg into the open air at a considerable velocity. It is often overlooked that in calculating the effective length of the lower leg of a siphon, the velocity head of the issuing water has to be deducted from the theoretical head of 33.9 ft. This fact is carefully considered by designers of draft-tubes for turbines, and such tubes are enlarged somewhat toward the outlet end, in order to decrease the velocity of the water and the corresponding velocity head. It is impossible to realize the full theoretical amount of suction, and the practical suction head for draft-tubes is generally limited to about 20 ft. of water column.

Assume, for example, a siphon having an effective head of $h = 25.0$ ft., and an efficiency of 60%, so that the velocity of the water in the siphon would be:

$$v = 0.60\ \sqrt{2\,g\,h} = 0.60\ \sqrt{2 \times 32.16 \times 25.0} = 24.0 \text{ ft. per sec.}$$

The corresponding velocity head at the outlet is,

$$\Delta h = \frac{v^2}{2\,g} = \frac{24.0^2}{2 \times 32.16} = 9.0 \text{ ft.}$$

Consequently,

$$h + \Delta h = 25.0 + 9.0 = 34.0 \text{ ft.}$$

This corresponds to practically the theoretical maximum atmospheric pressure at sea level, and, therefore, a suction head of 25 ft. appears to be about the limit for true siphonic action even under the most favorable circumstances.

In practice, a certain allowance has to be made for altitude and the daily and seasonal variations of the atmospheric pressure. It appears, therefore, that the safe limit of the length of the lower leg of a siphon which discharges freely into the open air is about 20 to 22 ft. For siphons with a submerged outlet the conditions are somewhat more favorable, and calculations will show, in every case, the limit for true siphonic action.

In order to illustrate these conditions more clearly, assume a siphon of constant cross-section with a submerged outlet and a lower leg the length of which is considerably in excess of 33.9 ft. Assume, further, the flow to have been interrupted by the closing of a gate near the throat of the siphon. At sea level, the air pressure on the lower pond (or the vacuum at the upper end of the pipe), would then keep in equilibrium a vertical water column 33.9 ft. high.

Now, assume, the gate near the throat to be opened somewhat, such that a limited quantity of water is admitted, producing at the outlet of the pipe a flow of the velocity, v. This corresponds to a velocity head, $x = \frac{v^2}{2\,g}$, which is lost, and the top of the water column will be lowered by the same amount, x.

According to Bernoulli's theorem, the pressure head plus the velocity head equals the static head at every point of a conduit, if friction, etc., is neglected. The static head in this case can never exceed the outside air pressure (vacuum at top of pipe).

For the conditions stated, $H + x = 33.9$, in which, H equals the height of the water column in the lower leg of the siphon if the flow, v, occurs. Consequently, as x is always positive, $H = 33.9 - x$. In other words, for a siphon of uniform cross-section and a length of lower leg of 33.9 ft., a vacuum would be likely to occur near the throat, thus causing occasional breaks of the water column and intermittent flow.

If the lower legs of such siphon spillways are made longer than the previous calculations would indicate to be safe, the water column is likely to "break" frequently, thus leading to intermittent flow and heavy shocks and vibrations which are known to accompany the abrupt changes of flow of water. The writer, therefore, is unable to agree with the provisions made, for instance, for the Alpine Dam and the Hetch Hetchy Dam for which the vertical distance between top of siphon and outlet is more than 30 ft. The length of the lower leg in excess of the limit for true siphonic action is not only useless as regards siphoning, but, in certain cases, it may be objectionable on account of the vibrations which result from occasional breaks of the water column. In addition, it involves unnecessary expense and is likely to lead to over-estimating the capacity of such spillways.

Although the formulas for calculating the discharge of siphons are simple, if the efficiency coefficient is known, the writer has found a graphical presentation of these equations helpful for quickly estimating the cross-section, length, and economical height of siphon spillways for any given amount of discharge, etc.

Robert E. Horton,* M. Am. Soc. C. E. (by letter).—The utility of a siphon spillway depends, among other things, on the friction losses which it entails or on the coefficient of discharge through the siphon. Experimental data for friction losses in siphons are meager. The results of experiments by Weirich on a small model siphon have recently been published† and are summarized in Table 10. These experiments show coefficients of discharge of about 97%, which are surprisingly large and appear to require some explanation not given in the original paper. The experiments, however, relate to nearly ideal conditions, the sections of the siphon being shown in Figs. 20 and 21.

Head losses in automatic siphons may be considered in three parts: (*a*) Entry head; (*b*) bend loss; and, (*c*) pipe friction in the siphon leg.

As the bend loss is usually an important factor, some discussion of the nature of such losses due to curves in pipes appears to be relevant. The bend loss proper may be considered as that loss of head due to eddies or shock, which is occasioned solely by change of direction. The question arises, what is the nature of the variation in this loss with the radius of the bend? Experimental data are not altogether consistent, partly, no doubt, in some cases,

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† *Annales*, Direction General of Forests and Waters, Minister of Agriculture, France, Fasicule 49, Paris (1917–18), pp. 570–572.

through failure to separate the true bend loss from the accompanying pipe losses. The general indication seems to be that the bend losses have a minimum value for a bend of a certain radius.

TABLE 10.—WEIRICH'S EXPERIMENTS ON SIPHON SPILLWAYS.

Number of experiment.	Upper level, in meters.	Lower level, in meters.	Fall, h, in meters.	Theoretical discharge, Qt, in cubic meters.	Actual discharge, Qw, in cubic meters.	Discharge coefficient, c.
1	+0.002 (−0.448)	−0.409	0.412	0.009	0.00861	0.956
2	−0.037	−0.409 (−0.448)	0.372	0.00854	0.00819	0.958
3	± 0	−0.448	0.448	0.00939	0.00916	0.974
4	± 0	−0.448	0.448	0.00939	0.00914	0.975
5	+0.006	−0.450	0.457	0.00947	0.00932	0.984
6	+0.004	−0.450	0.455	0.00945	0.00921	0.975
7	+0.000	−0.450	0.450	0.00940	0.00912	0.970
8	−0.011	−0.450	0.439	0.00928	0.00901	0.971
9	−0.012	−0.450	0.438	0.00927	0.00906	0.977

Experiments are usually performed on a pipe system involving both the bend and antecedent and subsequent sections of tangent. If the system contains a given or fixed length of straight and curved pipe, the pipe friction proper will be independent of the radius of the bend and will be constant. It follows that under these conditions a minimum total head loss for the system with a given radius of bend can occur only in case the loss due to change of direction alone is a minimum for some particular radius. The question may pertinently be asked, why should the loss due to change of direction have a minimum value for some particular radius and why should it not continue to decrease as the bend radius increases? Some light may be thrown on this matter by hydrodynamic considerations. It appears from experiments on ship profiles and bridge-pier models that there may be less formation of eddies for a surface curved to conform closely to the natural curve of stream-line flow in passing an obstruction than for any other form of surface. Thus, the well known "fishhead" profile may produce less eddy loss than either a curve of smaller or larger radius. If the action of water in passing around a bend in a pipe is analagous to the case mentioned, it is evident that a pipe of a certain radius may conform more nearly with the natural stream-line profile of the water when forced to make a change of direction, than for any other radius and thus yield less shock loss. Assuming this to be the case, it can easily be shown that the radius for minimum total losses, including a bend in a pipe system, may have three different values, depending on the precise losses under consideration. The precise law of relation of shock loss to radius of the bend is not known, but it may be assumed to be some function, $f(r)$, of the radius of the bend, which function gives a minimum value, $K_1 \frac{v^2}{2g}$, of the shock loss for some particular bend radius, r_1. Consider, first, the total head losses with reference to the bent part, $a\,b$ (Fig. 22) of a pipe of radius, r. The pipe friction proper is $K_2 \frac{L\,v^2}{2\,g}$, in which, $L = \frac{2\,\pi\,\theta}{360}\,r$.

The total head loss, due both to friction and change of direction between a and b, is,

$$F = \left(\frac{2\,\pi\,\theta}{360}\,r + K_1\,f\,(r)\right)\frac{v^2}{2\,g}$$

For a given central angle, θ, the pipe length, L, and the corresponding friction vary as the radius increases. Since $f\,(r)$ is probably not a simple linear function of r, it is evident that the value of r, which will give a minimum value of f in this case, say, r_2, will be different from the value, r_1, corresponding to a minimum loss due to change of direction alone.

Next, consider the system (Fig. 23) comprising two sections of tangent and an intermediate bend. The total length of pipe, straight and curved, in the system, decreases as the radius, r, increases. In the former case, the pipe friction proper was an increasing function of the radius, whereas, in the latter, it is a decreasing function, therefore, it is evident that the function for the total friction loss in the system will have a minimum for some larger value of r than in the preceding case. This last case is the one encountered in practice. The relations between curve loss proper and total loss in the three cases are illustrated in Fig. 24, in which Diagram (a) represents the loss due to change of direction; Diagram (b) the total loss in bend of constant angle and varying radius; and Diagram (c), the total loss in a system connecting the points, $a\,d$ (Fig. 23), and containing a bend of varying radius.

The author has not discussed the relative utility of siphon spillways and other forms of reservoir outlets for different conditions. In view of the increasing interest in the use of reservoirs for flood control, and the intimate relation of the size and type of outlets to the cost and utility of such reservoirs, the following three illustrations are presented as more or less typical of the results obtainable with different types of outlets on the control of a given flood. For example, assume a triangular flood graph with an assumed inflow rate rising from zero to 10 000 sec-ft. in 24 hours and then subsiding at one-half the rate of rise. Let it be required to determine the maximum stage in a reservoir of an area of 500 acres, a depth of 25 ft. at the dam, and a volume of prism storage equal to 5 000 acre-ft., and also to determine the maximum outflow rate, in cubic feet per second, for each of the following conditions:

(1) Reservoir with a permanent outlet tunnel of a capacity of 5 000 sec-ft. at the spillway level and a spillway 100 ft. long; the reservoir initially empty.

(2) Reservoir with siphon spillways of 5 000 sec-ft. capacity and a surface spillway 100 ft. long, the siphon priming level being 2 ft. below the spillway crest and the siphons arranged to draw the water 5 ft. lower, or 7 ft. below the crest, before ceasing operation, the reservoir initially full nearly to priming level and containing 4 000 acre-ft. of previously stored water.

The graphs of inflow rate, reservoir level, and outflow rate for the two conditions have been determined and are given in Fig. 25, which shows that the maximum reservoir stage and maximum outflow rates in the two cases are substantially the same. The assumption of equal outlet and spillway

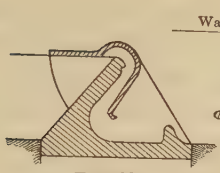

Fig. 20

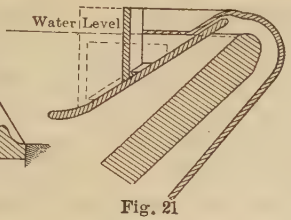

Fig. 21

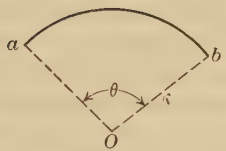

Fig 22

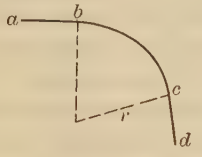

Fig. 23

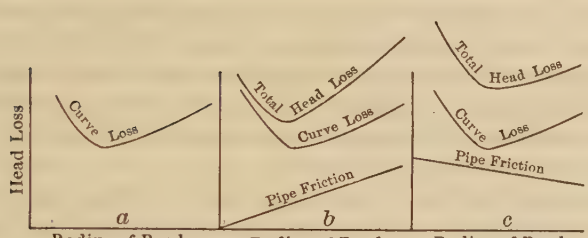

Fig. 24

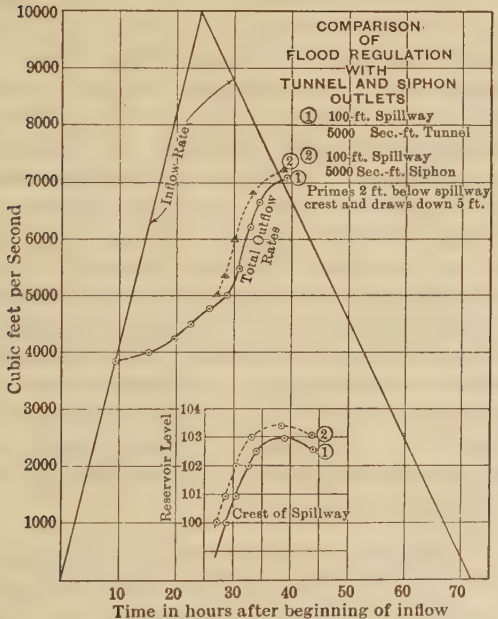

Fig. 25

capacities for both reservoirs is, of course, arbitrary and was made to insure the identity of the comparison. A better regulation with the siphon spillway might be obtained with a somewhat different combination of siphon capacity and spillway length without material difference in cost.

The comparison serves to show that for reservoirs with equal flowage areas and equal prism capacities, with dams of equal height, length, and section, and, therefore, at practically identical cost, substantially the same degree of flood regulation can be obtained with siphon spillways as with a permanent outlet tunnel, even though the reservoir is initially full nearly to spillway level in the case of the former, and is entirely empty in the case of the latter. Both reservoirs are entirely automatic in operation. It is evidently possible by means of siphon spillways to devise a reservoir that will afford a degree of automatic flood regulation equal to that obtainable by permanent outlet tunnels for any given flood, and, at the same time, utilize a major portion (four-fifths in this case) of the reservoir prism for storage for other purposes. It is true that the reservoir with siphon spillways may not give as good spillway regulation when initially empty as when initially full, as in the latter case the siphons will not come into operation until later. However, a reservoir with siphons can be designed to equal the permanent outlet tunnel for almost any condition, by the proper adjustment of the siphon capacity and priming levels for the different siphons.

The argument that part of the flowage area of a dam with a tunnel outlet need not be taken in fee and may be used precariously for agriculture has merit only in certain cases. Obviously, buildings must be removed and either an easement for occasional flooding obtained or else damages must be paid when they occur, if a permanent outlet tunnel is used. In any event, there will be a material reduction in the market value of the entire flowage and, therefore, in the wealth of the community, which is properly chargeable to the cost of flood-protection works. However, where a reservoir is constructed in rough or mountainous country, the flowage may have practically no value, whereas the prism storage, if it can be withheld and rendered useful when needed, may be of great value for water-power development, or other uses. It is an obvious error to build a reservoir for flood protection only and maintain permanent open tunnel outlets therein, keeping it useless for all other purposes, when equally good flood protection can be obtained at the same cost by a nearly identical reservoir, capable, perhaps, of paying for itself through the value of the prism storage it provides. It is apparent that types of dam construction and kinds of outlets best adapted for flood protection in different cases may differ widely. Siphon spillways seem to merit serious consideration for use in cases to which a careful analysis shows them to be adaptable. The analysis should be made, as without it, neither their use nor their rejection is justified.

A. Griffin,* M. Am. Soc. C. E. (by letter).—The writer will limit his discussion to the siphons of the Ripon Spillway, illustrated by Fig. 6.

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These siphons were designed and built by the writer in 1915 for the South San Joaquin Irrigation District, Manteca, Calif. The design was submitted to Mr. Stickney, and, at his suggestion, an alteration was made in the position of the air duct connecting the priming siphon and the main siphon.

The purpose of the siphon is automatically to control variations in the flow of the Main Irrigation Canal of the District, in order to maintain a constant water level at the terminus of the canal. This canal is about 23 miles long, with a maximum capacity of about 700 sec-ft.

In the lower 16 miles, the total flow is distributed to eighteen laterals, varying in capacity from 33 to 125 sec-ft. The Ripon Spillway is located where the main canal terminates by dividing into two laterals, each with a capacity of about 125 sec-ft. The capacity of the main canal diminishes as successive laterals take off, but the capacity at any point is considerably in excess of the combined capacities of the laterals at and below that point. By means of automatic radial (Taintor) gates, the surface of the water in the main canal is held at a fixed and predetermined elevation at the take-off of the laterals, regardless of the quantity of water flowing. For various reasons, the laterals have their flow altered from time to time and these alterations are made by raising or lowering the intake gates from the main canal. The elevation of the water, however, is maintained unchanged at the head-gate of the lateral in question and at the head-gates of all laterals farther down stream, by the automatic radial gates. Any alteration in the flow of the main canal is then transmitted to the end of the canal, where the Ripon Spillway is located. At this point, the alteration is discharged by the automatic siphon spillways, if it is an increase, or it is supplied to the main canal by a reduction in the flow of the siphons, if it is a decrease. In addition to variations in flow, owing to changes in the flows into various laterals, there are also regular and irregular variations from a number of other causes which cannot be eliminated.

The principal function of the siphons is to dispose of increases in the flow and prevent the increase from accumulating and topping the bank or increasing the flow in the laterals at the terminus to the danger point. Naturally, the siphons control the variations only within certain limits, and they are supplemented by a manually controlled spill-gate; there is also frequent attendance by the ditch-tender who lives at this location. The service performed by the siphons could be performed by an automatic radial gate, but not so satisfactorily. One serious objection to the automatic radial gate as a spillway is that it is difficult or impossible to prevent a certain amount of leakage when it is closed, whereas the siphon has no leakage. The automatic radial gate is quite satisfactory where water is always passing it and where it is only necessary to maintain a constant elevation, as the flow fluctuates. The gate has a wider range of application, but the siphon has the advantage of being free of moving parts and, within the sphere of its application, it is completely automatic and practically "fool-proof."

The conditions to be met by, and the services required of, the Ripon Spillway are as follows:

1.—A possible fluctuation of 200 sec-ft. in the flow in the main canal during the absence of the ditch-tender.

2.—The requirement that this fluctuation in flow (if an increase) be disposed of by spilling it into the Stanislaus River, and if a decrease that it be supplied by reducing the quantity then spilling.

3.—There is the further requirement that the elevation of water in the main canal shall be subjected to the least possible variation and especially that it shall not rise above a predetermined level. The canal is in high fill in this section and, due to burrowing animals and other causes, there is great risk in permitting the water to rise above the elevation at which it is normally carried.

4.—It is also desirable that there should be no loss of water by leakage, when there is no surplus.

There are two siphons, each of a capacity of about 100 cu. ft. per sec., symmetrically arranged, one on each side of a manually operated, radial spillway gate, which is essential for other purposes. One siphon would have served the purpose, and the principal consideration in building two was the appearance of the structure. The elevation at which the siphons will prime fully and discharge their full capacity is less than 0.2 ft. above the elevation at which the priming will break and the flow of water cease. The siphons will discharge automatically any fractional amount of their full capacity, equal to the surplus flow requiring to be spilled at intermediate elevations, and will discharge a mixture of air and water when not flowing at full capacity. The time required for priming is not important in this case. The siphons will prime in less than 5 min., which is less than would be required for any possible increase of flow to raise the water to a dangerous elevation. The writer has no information as to whether the time of priming is constant for all rates of rise in the water level.

As originally built, the siphons were as shown in Fig. 6. It will be noted that, on the up-stream side, the priming siphon extends down several inches below the crest. This caused the priming siphon to continue operating at part capacity after the main siphon had stopped flowing. This flow did no particular harm, but wasted a small quantity of water and caused visitors to ask embarrassing questions; therefore, at the first opportunity, it was cut back so that the intake to the priming siphon is horizontal and its position is on the same level as the crest. This permits both the large and the small siphons to break their priming at the same time.

Many engineers have asked how the priming siphon was constructed. This was done by casting the partition wall above the point where the space became too narrow to remove forms as a unit and by setting it in place after the main crest wall had been poured. This unit projected several inches into the side-walls and was embedded, and the joints were made air-tight when the side-walls were poured. At one time, it was proposed to make an inside form of mud or moulder's sand and remove this by a jet of water. The vertical part of the priming siphon connects into a sewer pipe embedded in the concrete, which leads to one side of the main siphon and back into it at a point where the air that is pumped out, will not get back into the vertical leg of the main siphon. The flow of water through the priming siphon seals the lower leg of the main siphon.

The question may be asked whether the priming siphon was necessary, or whether the same closeness of regulation could have been obtained without it? The writer frankly admits that he does not know. There are many ways in which the siphons might have been designed, and it is possible that some design might have given the same results with a less complicated arrangement. Extremely close regulation was necessary; the design adopted seemed to promise this and the results have fully justified the expectations.

Floating material is kept from the air vent by a baffle in front of the siphons. There seems to be no tendency for the priming siphon to choke with floating material, and, in fact, there is little opportunity for débris to reach it.

The writer believes that there is an important field for the use of the automatic siphon spillway in irrigation works, in giving, automatically, various controls over fluctuations in the flow of canals, which in actual operation it is not possible to eliminate. Many engineers are not familiar with the siphon and the various services that it can be designed to perform, and probably many of them feel (as the writer did at one time) that concrete is not a satisfactory material for building siphons. There need be no apprehensions on this point.

It appears to the writer that a useful type of siphon could be designed, which could be primed at any water level by a manually controlled jet-pump or other suitable device and which would continue in operation until the priming was broken by opening an air-inlet valve. Such a siphon would require little effort to start or to stop, as compared with the ordinary types of gates, and it would have the distinct advantage that there would be no leakage when not in use.

It would be useful to the Profession if detailed studies of the behavior of siphons of various designs were made by those familiar with their operation, and the results published.

E. S. Lindley,* M. Am. Soc. C. E. (by letter).—This paper gives valuable information about a device that has most useful applications, as long as its limitations are understood and allowance is made for them. Its novelty and ingenuity make many engineers chary of utilizing it, but the writer has seen it used, for example, where a failure would involve a breach in a supply canal directly above a hydro-electric generating station, 100 ft. or more below.

The writer has seen a siphon of the Gregotti design, as shown in Fig. 11, operating under a head of 14 ft. This siphon and one sluice-gate were passing the discharge of a canal and maintaining a level from 3 to 5 cm. below the lip and crest of the siphon. As a demonstration, a second gate was opened until the flow of the siphon was broken, and then adjusted until the canal was at the same level. The gate was then closed and within 30 sec., the siphon was again flowing, the maximum rise having been 2 cm. above the lip, and the canal level was then again drawn down to the former level; it could be heard loudly sucking, and could be seen sucking the tops off approaching wavelets.

That experience leads to the inquiry as to what area of air vent is necessary effectually to break the siphon action under different heads and circumstances?

Another point on which information is needed, is the ability of materials to withstand the action of high velocities around bends. The following is

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quoted from the authors' reply to discussion on a paper* by Sir Murdoch Macdonald and Dr. Hurst before the Institution of Civil Engineers, in reply to the writer's inquiry as to the effect of velocities up to 70 ft. per sec.:

"At first a distinct roughening of the picked dressed surface was apparent. Consideration showed that this was the result of the crushed and disturbed particles, at the points where the tool struck when the stone was being dressed, being loosened, and removed by the running water. When these particles were washed away, no further action was discernible over the 19 years during which the dam had been used, except at a point where, owing to the conformation of the sluice and the fact that the iron baffle-plate at the gate recess did not reach to the bottom, a stream of water impinged directly on to granite ashlar and eroded it to some extent. The completion of the baffle-plate put an end to this action. In the iron sluices also there was no erosion."

Table 1 shows a consistent set of tests, but a diagram of the siphon tested is needed to make them useful. The tests quoted by Mr. Wegmann in Table 5, although apparently for siphons of the same form, vary so greatly that even the mean value is less than the former tests.

Table 6 shows that deeper submergence of the outlet adversely affects the efficiency of starting; judging by what has been stated previously by the writer, the Gregotti design is a more efficient starter. Mr. Creager's point that the total head is not limited to the height of the water barometer, is worth keeping in mind; but the suggestion to utilize more head by gradually contracting the down-fall pipe would seem to have little application, as only by converting the excess drop that is not used in friction into velocity can a break of water column be avoided.

On page 1126, Mr. Creager states that "this time [required to remove the air trapped in the throat] is proportional to the average flow during the operation of priming"; *prima facie,* the rate of removal of air is proportional to the flow, so the time should be inversely proportional to the flow: Is this rough conception erroneous?

The author quotes one instance of a lock filled and emptied by siphons. Another instance is that of a lock allowing communication between the river and the cross-country canal at Minden, Germany. A lock, 85 by 12 m., with 6-m. difference of level, is operated by pairs of siphons, 2.0 by 0.8 m., at the head and at the foot. For each pair, a chamber, 3½ by 3½ by 2 m., is provided, the air-tubes being 20 cm. and the water-tubes 40 cm. The siphon crests are level with the upper water at each end. When the writer saw these siphons they were under construction (1914), so no figures of performance can be quoted.

Another device of the siphon type, although not a vacuum siphon, is intended to avoid accumulations of air, in falling pipe conduits. This apparatus† was devised by the late Sir Willam H. Lindley for the Baku Water-Works. The pipe is laid in sections on slightly rising gradients, in which the flow of water is in the direction of the tendency of the air to rise, and it is carried along. Drops are effected by short lengths of vertical pipe of a diameter so much smaller that the velocity will carry the air down. The top and bottom

* *Minutes of Proceedings,* Inst. C. E., Paper 4350.

† *Proceedings,* Annual Meeting of the Deutscher Verein von Gas- und Wasserfachmännern, 1909.

bends are made by transitions that avoid excessive friction losses and prevent lodgment of air.

J. T. Noble Anderson,* M. Am. Soc. C. E. (by letter).—This paper marks a step in the onward progress of the science of Hydraulic Engineering. One phase of this question, apparently not yet dealt with, came under the writer's notice about four years ago at Canberra, the Federal capital of Australasia, and was novel in some respects.

The chief artificial feature of Mr. Walter Burleigh Griffin's design for the beautification of Canberra is to turn the Molonglo River into two beautiful lakes. The combined area of these lakes would be 3 500 acres, and the catchment area discharging from them is slightly less than 800 sq. miles of undulating and, in places, almost mountainous country, with a recorded maximum rainfall of slightly more than 4 in. in 24 hours. The low-river flow is barely sufficient to offset the evaporation from the surface of the proposed lakes, whereas the calculations show that provision must be made for floods of 30 000 sec-ft., and in case of extreme floods, probably 40 000 sec-ft. should be expected.

The main feature of the problem was to provide automatically that the range of surface level in the lower lake should not exceed about 2 ft. 3 in. In the upper lake which, contrary to the usual experience, is the flatter portion of the river, the most serious problem would be the disposal of silt; this problem will also be serious in the lower lake.

Before designing the siphons for these lakes, the writer constructed large experimental siphons and, by a rough check over a gauging weir below and the measured area of a reservoir above, he made a series of experiments. The points investigated in these experiments were:

1.—What amount of rise above the crest under various conditions was sufficient to start siphonic action?
2.—What amount of fall below this level sufficed to stop siphonic action?
3.—How much could the efficiency be increased by the use of scientific stream-line design?
4.—To what extent could better velocities than that due to atmospheric pressure be obtained by giving a greater fall in the siphon?

For these experiments, the siphon was constructed with its cowl and throat of ¼-in., riveted, steel plates, with a concrete and timber inlet. The area was varied in order to give approved *vena contracta* results, the minimum area, at the throat, being 4 ft. wide by 2 ft. 9 in. deep. To control the vacuum, it was fitted with a number of 1-in. valve connections, by which, also, the vacuum at the crest and the pressures lower down the discharge leg were indicated by an ordinary Crosby steam engine indicator having a very light spring (¼ in. per lb.). The vacuum was also regulated at will by the introduction through the mouth of a 6-in., reinforced, rubber hose. One end of this hose was held by a mechanism hung over the water, so that it could, at will, be made to draw either air or water, whereas the other end was introduced into the throat of the siphon by a wire, which enabled its position to be controlled and determined.

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When this rubber hose was submerged and unable to break the vacuum, the fact was discovered that the siphon was capable of continuing action when the water in the reservoir had been lowered to 2½ ft. below the top of the mouth. The total area of the throat was 18 sq. ft., of which 10 sq. ft. was drawing air. The coefficient of discharge was 40 per cent. The importance of this discovery was that unless a new arrangement was introduced, the siphon would empty the reservoir to 2½ ft. before it stopped working, and with larger siphons probably more.

As floods in dry weather are not unknown, it was obvious that some contrivance must be found to prevent this heavy removal of water from the lakes. The use of the hose suggested a double siphon arrangement. Using the hose, and other openings, it was found that the vacuum could be broken before the surface level had been lowered more than 8 in. below the crest.

The introduction of the hose also had a distinct effect in moderating the surging and the rhythmic beating which is so marked in the action of these siphons. It evidently reduced the whirl and vortices, and as far as could be ascertained, had a beneficial effect on the coefficient of discharge. This fact suggests that some internal corrugations similar to the rifling of ordnance may be used with advantage in these siphons.

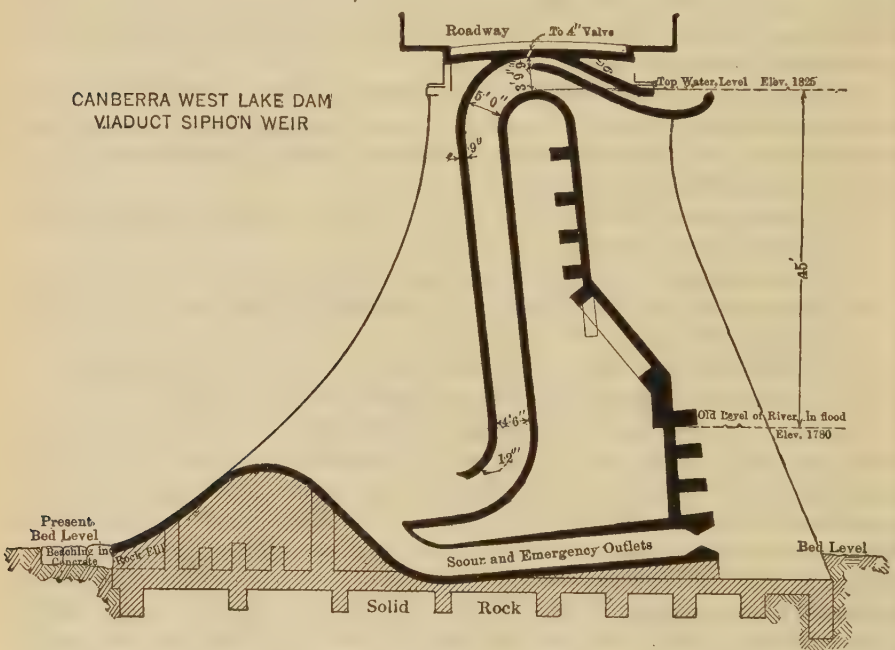

FIG. 26.

Fig. 26 shows the "stopping" siphons—so called to distinguish them from the type illustrated in Fig. 6—above the main siphons, which are designed to prevent the lake level being drawn more than 8 in. below the crest level. The under lip of this upper siphon is 6 in. below the level of the throat crest.

The dam was designed to be more than 70 ft. above the bed of the river, and the crest was 45 ft. above the highest ordinary flood level. It was designed to give a "drowned" discharge, notwithstanding the experiments indicated that, with the lower leg of the siphon inclined inward, there was no uncertainty about the starting. Often the full siphonic action followed the rise of the river within a few seconds after a level of 2 in. over the throat was recorded and a vacuum of 1¼ lb. per sq. in. When the volocity at the lower end (which was reduced in area), was equivalent to a head of 30 ft., the vacuum was nearly 8 lb. per sq. in., while considerable fluctuating pressure was recorded at the foot of the siphon, averaging 5 lb. when discharging through a drowned outlet about 36 ft. below the surface level of the dam.

The heavy vibrations accompanied by roaring noises, due apparently to the breaking of the column of water in the lower leg, were not noticed when the discharge was drowned. Probably a slight contraction from 18 to 12 sq. ft., was the cause of this favorable result.

From the experiments, it was estimated that these siphons would give not less than 60% efficiency under a head of 45 ft. A greater head could be obtained, but as the large floods for which the siphons were intended will always cause a drowned discharge, it was thought better to provide the heavy toe, as shown, and thus secure the seal for the siphon, and further insure a scour tunnel which could be artificially cleared by pumping, when silting became troublesome.

It was estimated that without using the scour valves these siphons alone could discharge 30 000 sec-ft. without causing the level of the lake to rise more than 1½ ft. above the crest. Greater floods than this could be met, to a great extent, by the further discharge from the fifteen automatic scour valves which would be open when this level had been attained. No attempt, however, was made to provide against unprecedented floods.

It is unfortunate that the funds provided for these experiments, although ample for the construction of a larger siphon (and the facilities for handling a large quantity of water were exceptionally good), would not admit of the construction of an accurate measuring weir. The quantities dealt with were far beyond the range of the weir previously constructed on the river. This gauge weir was also too far from the dam to give a sufficiently prompt or sensitive response to the experimental variations, and the result is that the records were of little value, except as a means of comparison. The writer believes that an efficiency of 60% was obtained and that accurate measurements would have shown considerably more than 70 per cent.

Owing to the aftermath of the World War, these lakes will not be constructed for some time, and the writer regrets that he cannot give the sequel to these tests which were conducted during 1917.

G. F. Stickney,* M. Am. Soc. C. E. (by letter).—In the various discussions of the siphon spillway, there appear to be some misconceptions of the siphonic action, which the writer will endeavor to explain and to show that a head, greater than the "siphonic limit" of 33.9 ft., cannot be used to increase the discharge, as proposed by Mr. Creager.

* Cons. Engr., Albany, N. Y.

The fundamental principle of siphonic action is that the flow is due to the atmospheric pressure on the upper water surface. It is obvious that whatever volume of water may be made to pass through the crown of the siphon will be carried away by the lower leg and, if the lower leg is arranged so that there is no resistance to flow through the crown, the siphonic flow will be a maximum. No extension, or modification, of the lower leg can increase this maximum flow, although a contraction of the lower leg, or the introduction of resistances therein, may reduce the discharge through the crown, if the effect of these resistances extends that far. The function of the lower leg is to eliminate resistances at the crown and also to provide a channel through which the water will escape.

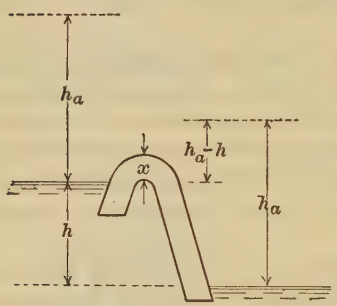

FIG. 27.

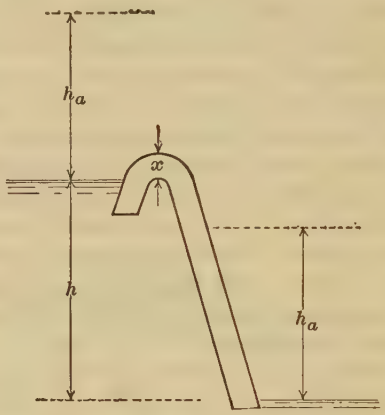

FIG. 28.

With this problem, absolute and not relative pressures are to be considered. There are two cases, as illustrated in Figs. 27 and 28. In Fig. 27, the head is less than the siphonic limit, whereas, in Fig. 28, it is greater.

Let h = the difference in elevation of the water surfaces, in feet.
h_a = the head of water, in feet, that would produce a pressure equivalent to the atmospheric pressure.
x = the height of the crown section, in feet.

The siphons are of uniform cross-sectional area throughout and, in order to simplify the analysis, the losses of head due to entry, friction, and curvature, are disregarded as they do not affect the general principles, although they do have great effect on the discharge capacity. In Fig. 27, assume the siphon to be closed at its lower end and filled with water. The internal pressure at the bottom of the lower leg would then be that due to a hydrostatic head of $h + h_a$ ft. and the internal pressure at the center of the crown would be that due to a hydrostatic head of $h_a - \frac{x}{2}$ ft. If the lower end is opened, there will be a flow of water through the siphon, and the internal pressure at the end of the lower leg must be the same as the external atmospheric pressure, which is represented by a head of h_a ft. In accordance with Bernouilli's theorem:

$$H_v + H_p = H_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which H_v = velocity head; H_p = pressure head; and H_s = static head.

From Equation (1), the velocity head at the bottom of the siphon may be found to be

$$H_v = (h + h_a) - h_a = h \text{ ft.}$$

and the velocity head at the center of the crown is

$$H_v = \left(h_a - \frac{x}{2}\right) - \left(h_a - h + \frac{x}{2}\right) = h \text{ ft.}$$

As these velocity heads are the same, the velocity of flow must be uniform throughout, and the lower leg will be completely filled with water. In the second case shown, Fig. 28, when there is no flow through the siphon, the pressure head at the bottom of the lower leg will be $h + h_a$ ft. and the pressure head at the center of the crown will be $h_a - \frac{x}{2}$ ft., as in the previous case. With water flowing through the siphon, the velocity head at the bottom, would be

$$H_v = (h + h_a) - h_a = h \text{ ft.}$$

The velocity head, at the center of the crown, cannot exceed $h_a - \frac{x}{2}$ ft., as this is the entire head above that point. As the velocity head at the crown is less than the velocity head at the bottom of the lower leg, the velocity of flow at the crown is less than the velocity of flow at the bottom of the siphon, and the lower leg will not be filled with water. As a result, air released from the flowing water, under the reduced pressure, would accumulate in the leg, causing the water column to break and thus stop the siphonic flow. In order to avoid such a condition, the lower leg must be restricted in length so that its vertical projection will not exceed h_a ft., or resistances must be provided in the lower leg, to insure that it will flow full.

In the case of the Gibswil Siphon, in Switzerland, described by Mr. Wegmann, which operates under a head of 52.48 ft., the lower leg is tapered from 31.5 in. in diameter at its upper end to 23.6 in. at the lower end, thus making the cross-sectional area inversely proportional to the velocity of flow, in order that the leg will be filled.

The discharge of this siphon is no greater than it would be if the lower leg was of uniform area throughout and terminated at the siphonic limit.

Mr. Creager's conclusion that the time required to prime a siphon, as indicated by the test of the Ocoee Spillway, varies with the rapidity of the rise in the water surface, is probably correct. The writer's assumption that the time of priming is a constant and is not affected by the rapidity of the rise, was based on a misinterpretation of the test data, as pointed out by Mr. Creager. The writer has not intended to convey the idea that siphons are, in all cases, to be preferred to every other type of regulating device, but is mindful of the fact that conditions vary to such an extent that no one contrivance can be of general use. Where a very large discharge capacity per linear foot of dam is required, sluice-gates of the Taintor or of the Stoney type are preferable to siphons.

It is believed, however, that on account of reliability of action, simplicity and permanence of construction, freedom from maintenance and operating

expense, close regulation of the water level, and large discharge capacity, the siphon is ideal for many uses. Mr. Dunham's suggestions, of various difficulties that might be apprehended, are all pertinent, and these difficulties, with others as well, have occurred to many engineers. Large drift, such as tree trunks or timbers too long to pass through the bends, would become obstructions in a siphon if they should enter but, as the inlet may be placed well below the water surface, floating matter will not be drawn into the conduit. Small siphons should have their inlets protected by screens, as the possibility of clogging is greater, but, ordinarily, large siphons will not require protection. The probability is rather remote that the discharge capacity of a spillway containing several units, will be seriously impaired by the siphons being clogged. Choking of the conduit by an accumulation of ice, has not, as far as the writer knows, ever occurred. Where the outlets are exposed to cold winds, it is conceivable that the siphons might act as chimneys, for although the inlets are submerged, the small air vents are not covered ordinarily and hence a draft of cold air might be created, that would freeze any water trickling through the crown. When the water rose high enough to prime the siphon, however, the rapid flow would cut away any ice that had formed.

The spillway at Whitehall, N. Y., is exposed to extreme cold in winter but, it has never failed to operate during periods of low temperature. The only trouble experienced is that on occasions the siphons have continued to discharge after the pond had been drawn below the normal level, which was caused by the formation of an ice tube on the air vent, as the water receded, that prevented the admission of air to break the siphonic action. This difficulty may be obviated by locating the air vent above the high-water level with a suitable valve, operated by a float, that will close when the water rises and open when the level recedes.

The cost of a siphon spillway is somewhat more than the cost of a fixed dam of the same length, as the siphon usually requires more concrete and some reinforcing metal and additional form work is necessary in the construction. A general comparison between the cost of siphons and of gates is hardly practicable, as there are too many variables to be taken into account.

Mr. Kennison's criticism, that a siphon spillway cannot be depended upon to discharge floods of uncertain magnitude, is true if he means floods of greater magnitude than the siphons can accommodate. There is a practical limit to the discharge capacity of such a structure, as there is of submerged gates or similar devices. This fact was brought out by the writer, with the caution that a spillway must be designed for the maximum flow to be expected. A siphon spillway will prevent excessive head on, or overtopping of, an important dam, if given sufficient capacity, and it will do this far better than an overflow weir of the same length. Stream flow is variable, swelling gradually during flood, until the peak is reached, and then diminishing slowly. The maximum flow lasts, usually, for only a brief time. A siphon spillway attains its maximum flow as soon as it becomes primed and, thereafter, discharges at a practically uniform rate. The discharge from an overflow spillway varies with the flood height and does not attain its maximum rate until the peak of the flood is reached. Thus, a siphon spillway

passes a much larger volume of water during the early stages of the flood than an overflow spillway of the same length and this prevents the storage of water that occurs before the latter becomes really effective. The writer agrees with Mr. Muckleston's observations that the design of a siphon spillway, or any other structure in which the principle of the water barometer is utilized, is not a simple matter and that there are pitfalls which may trap the unwary. This is true, however, because little attention has been given heretofore to problems of this nature and with a more general appreciation and use of siphons, the difficulties will disappear. The illustration of a spillway for the Lethbridge Northern Irrigation Project is an interesting example of a converging-diverging siphon with an auxiliary priming device to remove air liberated under the decreased pressure at the crown. It is not believed that the hydraulic ejector will be necessary to keep the spillway in operation, after the flow is started, although it will be of benefit in priming the siphon. The writer would expect the capacity of this spillway to be less than the 300 sec-ft. claimed for it as, apparently, no allowance has been made in the computations for the loss of head at entry and, further, it is thought that the resistances at bends have been under-estimated.

Mr. Noetzli's conclusion, that the safe limit of length for the lower leg of a siphon that discharges freely into the open air, is about 20 to 22 ft., is evidently in error, as there are siphons in successful operation under greater heads, such as the Ocoee Siphons, 26.5-ft. head (Fig. 8), and the Alpine Dam siphons, 31-ft. head. In the example given of a siphon with an effective head of 25 ft., having an efficiency of 60%, the velocity head of 9 ft., inadvertently, has been accounted for twice, in computing a total head of 34 ft. The Alpine Dam siphons are at an elevation of 610 ft. above sea level, where the atmospheric pressure is equivalent to an hydrostatic head of about 33 ft. They operate under a head of 31 ft., which is the maximum head available, as the crown height of the siphons is 3 ft., and when the reservoir level rises 1 ft., the top of the crown is 2 ft. above the water surface. This 2 ft. that the water must be lifted, is deducted from the 33-ft. total head, leaving a 31-ft. effective head.

The discharge through a siphon may be computed with the same degree of accuracy as is obtainable in hydraulic computations generally, that is, with a limit of error of, say, 5 per cent. In designing a siphon spillway, a form must be adopted that will insure the priming of the siphon and full flow in the lower leg. Efficiencies of 60 to 70% may be readily obtained with simple forms, and higher efficiencies are attainable by the use of more complicated forms. There are practical limitations, however, to the modifications that may be justified to obtain increased efficiency. A high-velocity coefficient, at a restricted part of the siphon, does not necessarily indicate high efficiency, as this is determined by the relation of the actual discharge to the theoretical discharge, through the outlet, or, what amounts to the same thing, is a comparison between the actual and the theoretical discharge velocities under the head available.

Mr. Horton's Table 10, giving the discharge coefficients obtained in experiments with a model siphon in France, is interesting in showing the high efficiencies attainable.

It is to be regretted, however, that full details of the experiments are not available, by which such surprising results have been secured. The example presented of flood regulation by a detention reservoir, with tunnel outlet, as compared with a storage reservoir, with siphon spillway outlet, illustrates admirably one of the many uses for which the siphon spillway may be selected to advantage.

Mr. Griffin's query, as to whether the same closeness of regulation of the water level of the canal of the South San Joaquin Irrigation District could have been obtained by a type of siphon different from that used in the Ripon Spillway, may be answered in the affirmative. A quick-acting siphon may be designed to operate when the water level above the spillway rises 0.2 ft., without any auxiliary priming device, if the height of the siphon is small (say, 12 in. or less). This gives a small cross-sectional area per foot width, however, and, in consequence, the discharge capacity will be small, necessitating a greater length of spillway than is required where a greater height is used. With the conditions existing at Ripon, where the water level rises slowly, there is ample time for priming to be effected, by exhausting the air from the main siphon before the limiting height of water is reached and this is accomplished by the priming siphon with a rise in the water level that is too small to produce a jet of sufficient size to prime the big siphon. In experiments, made by the writer, with a siphon 4 in. wide by 3 in. high, it was found that the siphon would prime when there was a depth of 1.4 in. of water flowing through the crown. A siphon may be primed at any water level, not exceeding about 32 ft. below the top of the crown, by a manually controlled jet-pump. Such a device might be used to advantage in place of a gate-valve to draw off water at a dam or from a reservoir. The crown of the siphon would be above the water level, so that there could be no loss by leakage. At the base of the dam a small jet-pump connected with the crown of the siphon would serve to put the siphon in operation. The flow could be stopped by admitting air to the crown through another pipe. The arrangement would be similar to that in the siphon lock at Oswego, N. Y., except that the jet-pump would be used in place of a vacuum tank. Little effort would be necessary to start and to stop the flow, which would be effected by manipulating small valves, instead of the great effort that is required to operate cumbersome gate hoists, such as are ordinarily used.

In reply to Mr. Lindley's inquiry, it may be said that no investigation has been made to determine the minimum area of vent required to break the siphonic action under different heads and circumstances. Air flows at a high velocity under a small difference in pressure, between the inlet and the outlet of the conduit, and a small conduit will admit a large volume of air. It has been found that an air vent, one-twenty-fourth of the cross-sectional area of the siphon, is sufficient to break the siphonic action, and that the siphonic flow will stop before the vent is entirely uncovered by the lowering of the upper water surface. Ordinarily, siphon spillways are built of concrete which, if of good quality, will withstand a high velocity of flow without injury. The bends will be more susceptible to erosion than the straight parts. In a siphon operating under the maximum siphonic head of 33.9 ft., the greatest velocity attainable is probably about 37 ft. per sec. If this velocity is considered in

excess of what the material will stand, linings of cast iron or other suitable material may be used, but, as yet, such protection has not been deemed necessary. The removal of air trapped in the crown of the siphon, during the period of priming, is effected by the stream of water flowing through the siphon. Apparently, this air is taken up, mainly by the surface of the stream, and, therefore, the rate of removal may be considered as roughly proportional to the velocity of flow rather than to the volume of flow.

Mr. Anderson's description of the proposed siphon spillway at Canberra, and of the experiments made as a basis for the design of the siphons, is quite interesting. Regulation of the levels of the Canberra Lakes and the discharge of floods with a fluctuation not to exceed 1 ft., instead of the 2 ft. 3-in. range assumed by Mr. Anderson, can be secured by properly locating the air vents of the siphons.

That the experimental siphon was capable of continuing action when the reservoir had been lowered 2½ ft. below the top of the mouth is understandable, if the upturned part of the mouth is intended, but the statement that 10 sq. ft. of the throat area, out of a total of 18 sq. ft., was drawing air, is incomprehensible, as a vent of less than 4% of the cross-sectional area of the siphon is sufficient to stop siphonic action. The so-called, "stopping" siphon, shown in Fig. 26, is nothing but a conduit to the crown of the siphon, that admits air whenever the water level is drawn down below its inlet. There is no advantage in using this long conduit, as a simple vent in the up-stream face of the siphon will serve equally as well. Ordinarily, the vents are located just at the normal water level so that they will be sealed when the water rises, and are fully opened at the normal stage.

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Paper No. 1501

THE RECONSTRUCTION OF THE BALTIMORE AND OHIO RAILROAD BRIDGE CROSSING THE ALLEGHENY RIVER, AT PITTSBURGH, PENNSYLVANIA

By Philip George Lang, Jr.,* M. Am. Soc. C. E.

With Discussion by Messrs. Edward W. Stearns and Philip George Lang, Jr.

Synopsis

This paper describes the reconstruction of the bridge of the Baltimore and Ohio Railroad Company, across the Allegheny River on the line of 33d Street, Pittsburgh, Pa.

Although the new structure which replaces a bridge built in 1884 at the same location, is not of unusual size, the necessity of maintaining uninterrupted railroad operation during the work, the requirement of the War Department that no interference with river traffic be occasioned, and local right-of-way conditions which restricted to narrow limits the available working space, render the reconstruction a matter of interest.

On September 1st, 1920, a 265-ft., double-track, through, riveted truss span, was rolled to permanent line at the old or low level, and, on December 20th, 1920, was jacked a vertical distance of approximately 15 ft., to a new high level, on which date, also, a 436 ft. 9-in., through truss span was rolled to permanent line at the new grade.

The bridge of the Pennsylvania System, crossing the Allegheny River on the line of 11th Street, was raised on February 22d, 1919, therefore, the Baltimore and Ohio Railroad Company's structure is the second to be elevated in accordance with the order of the War Department, requiring the raising of all bridges crossing the Allegheny River in the Pittsburgh District, and is the first reconstruction work completed under that order. This work has earned the commendation of the War Department, and the elimination of a dangerous grade crossing has received the favorable recognition of the local authorities.

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Historical

The Baltimore and Ohio Railroad enters Pittsburgh from the east and follows the north bank of the Monongahela River to the Smithfield Street Passenger Station. Through train movement for points west of Pittsburgh is by way of Laughlin Junction, across the city in a general northwesterly direction. The location of the Baltimore and Ohio Railroad within the Pittsburgh area is shown on Fig. 1.

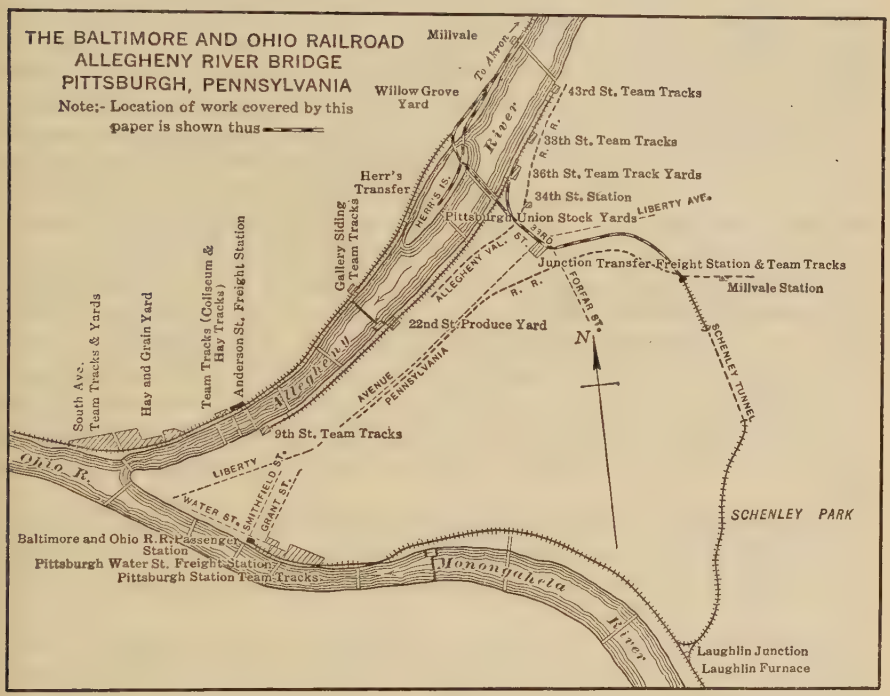


Fig. 1.

That part of the line from Laughlin Junction to Willow Grove is known as the Pittsburgh Junction Railroad and embraces what were originally parts of the contemplated trackage of two incorporations, namely, the Pittsburgh Local Railroad Company and the Pittsburgh Junction Railroad Company. As the Allegheny River Bridge played an important part in the development of the Baltimore and Ohio Railroad in and about Pittsburgh, a brief history should be of interest.

About 1880, the Connellsville and Pittsburgh Railroad, now a part of the Baltimore and Ohio System, which parallelled the right bank of the Youghiogheny River from McKeesport to Pittsburgh, had its terminus at Grant and Water Streets.

The Pittsburgh, New Castle, and Lake Erie Railroad, subsequently known as the Pittsburgh and Western Railroad and now a part of the Baltimore and Ohio System, was built in 1877 and 1878, and extended from Allegheny City to Harmony and Wurtemburg, Pa.

The Pittsburgh, Cleveland, and Toledo Railroad was opened to traffic in 1884. Its route extended through the Mahoning Valley, eastward, from Akron Junction to New Castle Junction. At the latter point, it connected with the Pittsburgh and Western Railroad, thus providing a through line westward from Allegheny City, now Pittsburgh North Side, to Akron and Cleveland Harbor, Ohio.

By this time the industrial development of the Pittsburgh Harbor District had assumed considerable importance. An ample supply of coal and coke was available, but ore in large quantities could be obtained only through the harbors on Lake Erie. This ore movement was handicapped by the inadequacy of the railroad facilities. The necessity for a direct Lake route connection was recognized and the Baltimore and Ohio Railroad Company was prompted to seek an affiliation with either the Pittsburgh and Lake Erie Railroad or the Pittsburgh and Western. A survey was made in 1880, with the object of securing a crossing of the Allegheny River at about the site of the present 7th Street Bridge and effecting a direct connection with the Pittsburgh and Western Railroad in the vicinity of its present passenger station. This project, known as the Pittsburgh Local Railroad, was incorporated on September 27th, 1880. The route as surveyed was unsatisfactory and other surveys were made in 1881. It was afterward suggested that the location of the Pittsburgh Local Railroad be changed from the Water Street Wharf to the present location of the Pittsburgh Junction Railroad. This arrangement was advisable, because the new route afforded a shorter, though more expensive, western outlet.

The Pittsburgh Junction Railroad was chartered on August 6th, 1881, and on October 1st, 1881, was consolidated with the Pittsburgh Local Railroad, the entire trackage of the two incorporations being designated as the Pittsburgh Junction Railroad.

It was decided to use 33d Street for the route of the Allegheny River crossing, and an ordinance of the City of Pittsburgh, dated July 30th, 1883. granted authority for a crossing of Liberty Avenue at grade. On March 1st, 1884, a contract was placed for the four double-track Main Channel spans, and on April 25th, 1884, the contract was placed for the three double-track Back Channel spans. The part of the structure on Herr's Island consisted mainly of embankment and fill. On April 25th, 1884, the contract was placed for the viaduct along 33d Street, which extended from Liberty Avenue to the first pier of the Allegheny River Bridge, a distance of 1 782 ft. The bridge which was completed and opened to traffic in 1884, served its purpose for more than thirty years.

The interval between 1884 and 1915 was marked by a steady increase in the weight of locomotives, with a corresponding severity of effect on bridges and roadbed. To meet these conditions, the structure was subjected to continual repair, careful maintenance, and alterations, in order to adapt it to the imposed demands.

For about ten years preceding the reconstruction of this bridge, it was known that the limit of economic possibility in the use of the structure had been reached. Negotiations which had covered a period of years, were finally

concluded on September 30th, 1915, when the Secretary of War issued a permit covering the reconstruction of the river spans.

The Plans of 1915

As contemplated in 1915, the plan for the structure provided for raising the track about 14 ft. for the main river spans, and the placing of a double-track, through truss span, 436 ft. in length, over the Main Channel, with a 265-ft., through truss span adjacent to and eastward thereof, and a through truss span across the Back Channel. The permit further stipulated that during the erection of the new bridge and the removal of the old structure, there be maintained between the two piers of the old Main Channel span, a clear opening of 150 ft. for the accommodation of river traffic. Falsework was to be placed for the remaining part of the Main Channel, as well as for the Back Channel, as necessary for the erection of the new structure.

The procedure contemplated was to erect the new channel spans on falsework, up stream from the existing structure, and to roll them to permanent line and elevation. The plans of the new structure provided a clearance of 51 ft. above Davis Island Pool, and a clearance of 44 ft. above the Herr's Island Pool which was 7 ft. higher than the Davis Island Pool. The center line of the two main-line tracks of the new structure coincides with that of the old, except that on Herr's Island, the tracks were to be re-located up stream from the previous location, the Herr's Island connection being placed immediately down stream from the new main-line tracks.

An ordinance was passed on April 3d, 1915, providing that the tracks of the Railroad Company along 33d Street should be carried on a viaduct supported on two lines of columns, one on each curb line, thus avoiding the dangers of placing any part of the support over the sewer in the bed of 33d Street.

The Liberty Avenue crossing provided by the new structure, consisted of a solid-floor, through, plate-girder span, about 66 ft. in length, and adjacent thereto, immediately eastward, a solid-floor double-track, through plate-girder span, approximately 62 ft. long, continuing in that direction with four spans of double-track, open timber floor, deck-plate girders. Crossing Penn Avenue and Smallman Street, double-track, solid-floor, deck plate-girder spans, 65 ft. in length, were placed. The tracks of the Allegheny Valley Railroad are crossed by a double-track, solid-floor, deck plate-girder span, 100 ft in length. The remainder of the structure consists of 30-ft. towers, with intervening spans of suitable length. The old and the new viaducts are shown in comparison on Plate XIII.

The entire structure was designed for Cooper's E-60 loading, 14-ft. track centers, and 16-ft. clearance diagram. In order that in the future the new structure might be adaptable to the new main spans over the Allegheny River, it was decided to construct the new viaduct so as to accommodate a 1½% descending grade, to be used until the reconstruction of the river spans, at which time a 0.3% grade would be obtained. The contemplated future raise of grade was taken care of by the use of additional pedestal blocks, where the stringers were placed on top of the cross-girders, and, at the points where

the stringers were framed into the cross-girders, by changing them to a position on top of the cross-girders, using suitable pedestal blocks. Where changing the stringers from framed-in to placing on top was not sufficient, arrangements were made to secure additional elevation by raising the cross-girders, the cross-girder connections, in this case, being detailed so that they could be readily elevated. In one double-track span, 30 ft. long, the girders could not be used, and new columns were also necessary at four of the bents, as well as additional miscellaneous material.

On May 31st, 1915, a contract was placed with the American Bridge Company, covering the fabrication and erection of the steelwork for the superstructure of the part of the structure from the abutment east of Liberty Avenue to the east bank of the Allegheny River. The total weight of steel entering into this viaduct was 7 354 238 lb.

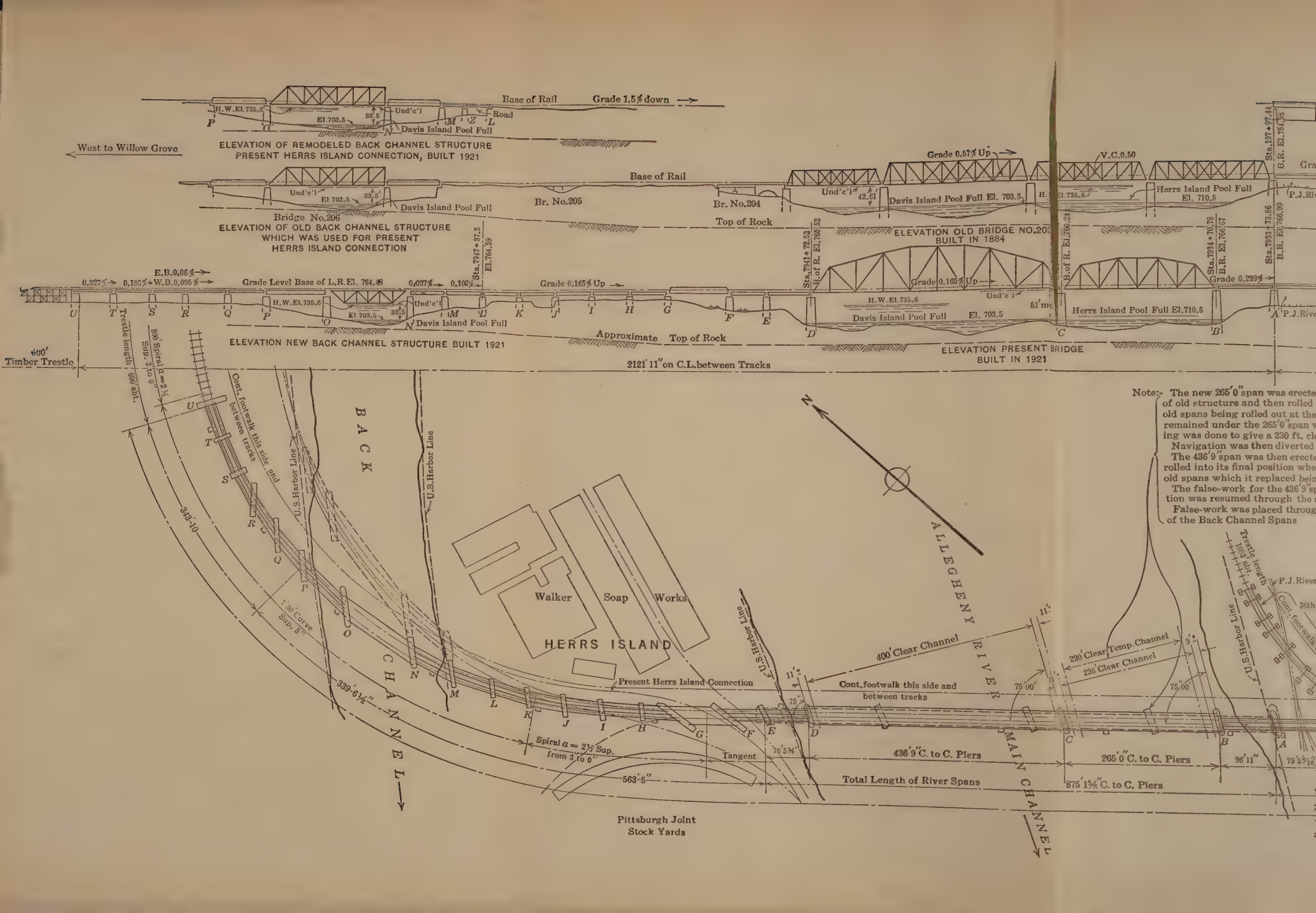
On August 12th, 1915, a contract was placed with P. J. Joyce and Company, Incorporated, of New York City, covering the masonry substructure for this viaduct, and the date of completion was set for June 20th, 1916. The Company commenced operations on September 13th, 1915. In the meantime, on August 14th, 1915, the City had placed a contract for the raising of the Herron Avenue (formerly Forfar Street) Bridge, with John Eichleay, Jr., who commenced operations on September 17th, 1915.

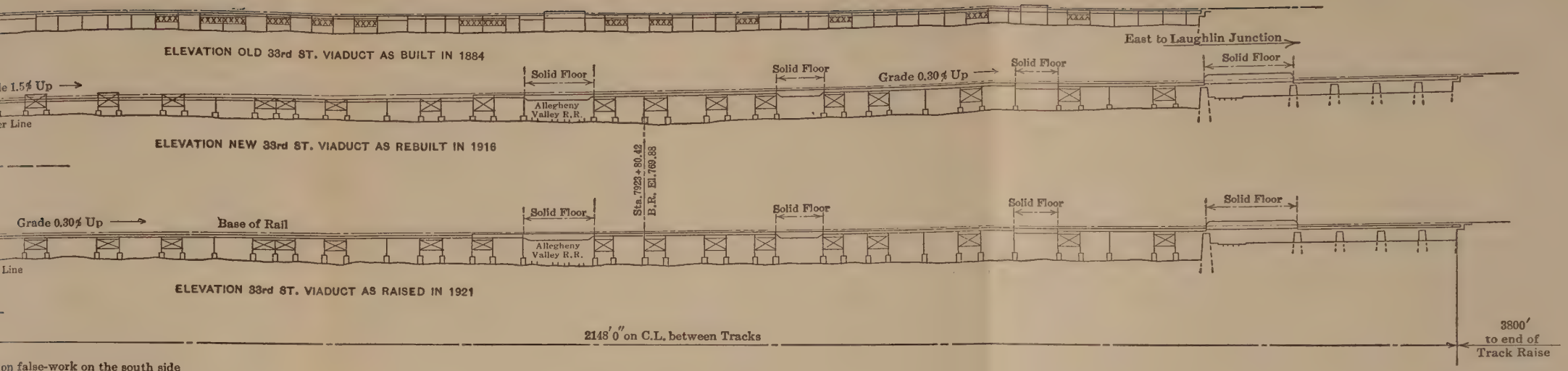
Construction of 33d Street Viaduct

In March, 1916, the forces of the American Bridge Company began the erection of the steelwork. It was necessary to consider the movement of traffic at this point, and make such adjustments as were needed to permit the handling of the reconstruction of the viaduct. Prior to beginning erection, single-track operation had been resorted to over this viaduct. It was deemed advisable, however, to divert all traffic to a temporary trestle constructed especially for this purpose. This trestle occupied, in general, the north sidewalk of 33d Street, between Liberty Avenue and the 36th Street Yard cross-over, and was built approximately to the new grade. This work was done in the spring and early summer of 1916, and traffic was transferred to the temporary high line on July 1st, 1916.

During the period in which the erection of this viaduct was in progress, the economic reaction of the World War was felt. The supply of labor was uncertain, the cost was rising, and it was necessary to concede several increases in the wage rate. On May 24th, 1916, the erection forces discontinued work, and remained idle until June 5th, 1916. During September, 1916, it was announced that, on all railroads, a basic 8-hour working day would become effective January 1st, 1917. In view of this fact, no precaution was spared to insure double-track operation over this viaduct by that date, and on October 16th, 1916, the east-bound track of the new viaduct was put into service. Traffic was placed over the west-bound track on October 22d, 1916.

Although a modern structure now extended to the east bank of the Allegheny River, its advantage was seriously impaired by the old and inadequate spans over the river. The supply of engines that could safely operate at this point had been reduced, and necessity demanded that the old spans,


ELEVATION OF REMODELED BACK CHANNEL STRUCTURE
PRESENT HERRS ISLAND CONNECTION, BUILT 1921
West to Willow Grove
Base of Rail
Grade 1.5% down
H.W.El.735.6
El.703.5
Und'c'l
Road
Davis Island Pool Full
Grade 0.57% Up
V.C.0.50
Br. No.205
Br. No.204
Herrs Island Pool Full El. 710.5
Bridge No.206
ELEVATION OF OLD BACK CHANNEL STRUCTURE
WHICH WAS USED FOR PRESENT
HERRS ISLAND CONNECTION
Top of Rock
ELEVATION OLD BRIDGE NO.203
BUILT IN 1884
E.B.0.06%
0.327% → 0.185% → W.B.0.095% →
Grade Level Base of L.R.El. 764.48
0.037% → 0.107%
Grade 0.165% Up
Grade 0.289%
Herrs Island Pool Full El.710.5
Approximate Top of Rock
ELEVATION NEW BACK CHANNEL STRUCTURE BUILT 1921
ELEVATION PRESENT BRIDGE
BUILT IN 1921
600' Timber Trestle
Trestle length 600 abt.
800' Spiral a = 2½ Sup. 3° to 0°
2121' 11" on C.L. between Tracks
Cont. footwalk this side and between tracks
U.S. Harbor Line
BACK CHANNEL
348'-10
339'-6½"
563'-5"
Walker Soap Works
HERRS ISLAND
Present Herrs Island Connection
Spiral a = 2½ Sup. from 3° to 0°
Tangent
Pittsburgh Joint Stock Yards
ALLEGHENY RIVER MAIN CHANNEL
400' Clear Channel
230' Clear Temp. Channel
236' Clear Channel
436'9" C. to C. Piers
265'0" C. to C. Piers
Total Length of River Spans
875' 1⅝" C. to C. Piers
Note:- The new 265'0" span was erecte
of old structure and then rolled
old spans being rolled out at the
remained under the 265'0" span w
ing was done to give a 230 ft. cl
Navigation was then diverted
The 436'9" span was then erecte
rolled into its final position whe
old spans which it replaced bein
The false-work for the 436'9" sp
tion was resumed through the
False-work was placed throug
of the Back Channel Spans



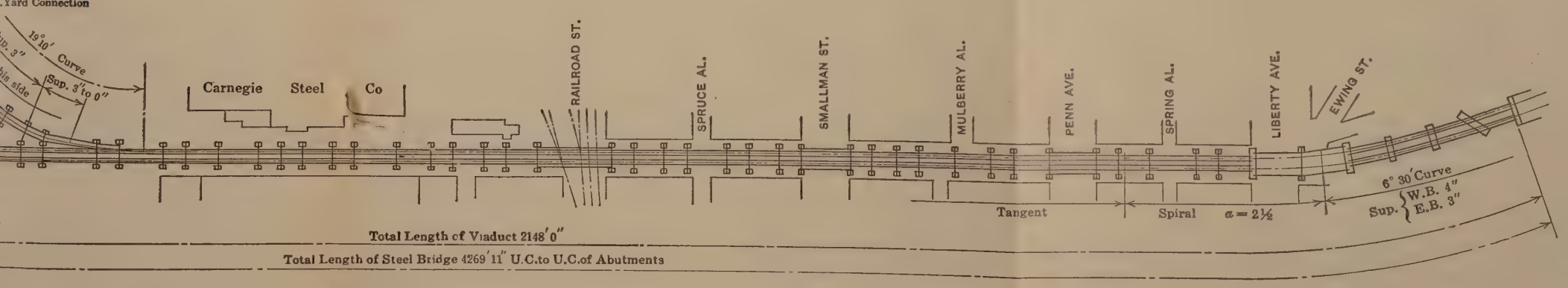
on false-work on the south side
to position when completed, the
ame time. The old pier which
s removed and sufficient dredg-
r channel under this span.
this new channel,
on false-work throughout and
the erection was completed, the
rolled out at the same time.
n was then removed and naviga-
in channel.
ut as necessary for the erection

ing

Yard Connection

THE BALTIMORE AND OHIO RAILROAD
ALLEGHENY RIVER BRIDGE
PITTSBURGH, PENNSYLVANIA
GENERAL PLAN

SCALE IN FEET
0 100 200 300 400



Total Length of Viaduct 2148′0″

Total Length of Steel Bridge 4269′11″ U.C. to U.C. of Abutments

during the remainder of their service, be utilized to the utmost; on October 23d, 1916, it became necessary to establish single-track operation over the river spans.

Plans for River Bridges

The reconstruction work on the 33d Street Viaduct had been followed with attention by the War Department, in view of its relation to the river spans and the fact that the viaduct and the river bridge actually constitute one homogeneous structure. On August 2d, 1916, the representatives of the Railroad Company were summoned to the United States Engineer's office, and questioned concerning the work on the river spans. It was then held by the Railroad Company that the work on the structure as a whole had been started, and that plans for the river bridges were in progress. The Railroad Company also stated that to undertake the reconstruction of the river spans prior to the completion of the 33d Street Viaduct was impracticable, because of the interference with traffic. The War Department extended the time limit one year, which provided for the commencement of the work not later than September 30th, 1917, and its completion within three years.

The Secretary of War demanded the raising of the Allegheny River bridges below Herr's Island, to provide a 47-ft. vertical clearance above pool full, and for those above that point, a clearance of 51 ft. was demanded. The statement was made that the bridges were an obstruction to navigation, and that immediate elevation was necessary. In view of the negotiations already under way between the War Department and the Baltimore and Ohio Railroad Company, and the contemplated renewal of its bridge, that Company was omitted from citation in the War Department order of March 23d, 1917, under which notices were served on the County of Allegheny and the Pennsylvania Railroad Company.

The Declaration of War with Germany by the United States made immediate traffic and operating needs a prime consideration, and work was suspended during the remainder of 1917 and during 1918. By rigid traffic restrictions and careful maintenance, the old bridge was kept in service.

With the termination of hostilities, the reconstruction of the river spans was again considered, and early in 1919 negotiations were resumed with the War Department. Certain changes were advisable, and on March 10th, 1919, a modified plan of reconstruction, submitted by the Railroad Company, was approved by the War Department. This approval was based on construction, as shown by Plate XIII.

The engineering problems in connection with this reconstruction were intensified by the demand of the War Department that the work be conducted so as to permit navigation on the river at all times, by the necessity of building the new structure at an elevation approximately 14 ft. above that of the old bridge, and by right-of-way conditions which necessitated the maintenance of traffic within the property lines.

The structure involved in this improvement has a length of 2 122 ft., and extends from the west end of the 33d Street Viaduct, at the east bank of the Main Channel, to the abutment at the westerly end of the new structure.

A series of girder spans were used for crossing the Island, and two truss spans, 265 ft. and 436 ft. 9 in., respectively, for crossing the Main Channel, thus providing clear channels for river traffic of 236 ft. and 400 ft., respectively, with a minimum overhead clearance above Davis Island Pool of 51 ft. The westerly approach consists of about 2 900 ft. of fill, and approximately 600 ft. of double-track, frame trestle supported on timber piles, between the fill and the west abutment of the main structure. The trestle is a temporary structure, and eventually will be replaced by an earth or slag fill. In general, the center line of the new structure coincides with that of the former bridge, except westward from about the center of Herr's Island, where there is a wide deviation. The fact that no traffic problem was involved in the construction of the west end of the new bridge greatly simplified the work.

Construction Features of New Bridge

The contract for the substructure was placed with the Vang Construction Company, of Cumberland, Md., on August 15th, 1919. Between the westerly end of the 33d Street Viaduct, indicated by Pier *A*, and the termination of the new steel bridge at Abutment *U*, Plate XIII, there are nineteen new concrete piers, designated by the letters, *B* to *T*, inclusive, occupying sites distinct from the old masonry, and their construction occasioned no interference, except at Pier *E*. The reconstruction of Pier *A* was not commenced until the new spans at that end had been placed, and then the new pier at this location was constructed on the old foundation which was in good condition and consisted of timber grillage on timber piles.

Piers *B*, *C*, and *D*, which support the 436 ft. 9-in. and the 265-ft. Main Channel spans, were built on timber caissons, sunk by the pneumatic process to footing on sandstone, about 63 ft. below the water level in the Herr's Island Pool. Borings were taken at the site of each pier prior to the commencement of this work.

The foundations for the Main Channel piers are from 14 to 16 ft. below what was at first thought to be suitable bearing strata. An unusual formation was discovered beneath the river bed. Below an apparently solid top course were thin layers of partly decomposed shale, separated by intervening mud strata. The caissons were nosed into the shale, and excavation was continued for a distance of from 14 to 16 ft. below the cutting edge.

The height to which these piers could be built was limited by the old bridge, and, in consequence, they were stopped temporarily at an elevation slightly below the under-clearance line of the old spans. The new spans were supported on steel columns, which were encased in concrete after the completion of the new superstructure to final line and grade.

The caissons used in the construction of these piers were built adjacent to the work, and consisted of timber boxes, 70 ft. long, 22 ft. wide, and 48 ft. high. Two muck shafts, 3 ft. in diameter, and one 3-ft. man shaft, were provided in each caisson. The side-walls of the working chambers were strengthened with reinforced concrete, and the roof of each consisted of a reinforced concrete slab. The combination of timber and reinforced concrete construc-

tion in such a manner as to provide sufficient buoyancy for the floating of the caissons to their sites, is worthy of note.

Pier *E* was located at the center of the arch of the old west arch abutment, and it was found that the area covered by the old arch was paved with two courses of stone masonry, supported on two layers of 12 by 12-in. timber and one layer of 4-in. planking, the entire area being supported by timber piles, spaced approximately 3 ft., center to center. This pier was sunk by means of a pneumatic caisson to a support on the old timber piles, about 17 ft. below the level of the Herr's Island Pool.

The piers on Herr's Island were footed on concrete piles, and as it was possible to detour the traffic on the single-track timber trestle, placed alongside the permanent alignment, these piers were built to final elevation before placing the steel. Piers *M* to *P*, inclusive, were footed on sandstone about 23 ft. below Herr's Island Pool. Pier *M* was built on a caisson, pneumatically sunk; the other three piers in this group were built by means of open coffer-dams, although, with Pier *N*, an air-bell, operated by pneumatic equipment, was used. Piers *Q*, *R*, *S*, *T*, and Abutment *U* are supported on concrete piles.

The conditions at Pier *N* (Fig. 2), which led to the use of an air-bell, were unusual. The depth from pool full to river bottom at this pier was about 12 ft., and the underlying soil consists of 4 ft. of sand and gravel, under which is a layer of shale or sandstone. With the object of reaching a sandstone foundation, the site was dredged to a depth of 18 ft., and a coffer-dam was constructed of a single row of interlocking sheet-piles which enclosed the pier and the adjacent area. The area inside the coffer-dam was dredged within 1 ft. of rock, and the excavated material was utilized for banking the walls of the coffer-dam.

When an attempt was made to drain the coffer-dam, considerable leakage occurred, being especially marked adjacent to the old masonry pier, to which the coffer-dam was joined by pockets filled with clay and sealed against the old masonry by tremie-deposited concrete. A diver had previously examined the old pier and it appeared to have been constructed on timber cribbing. On account of the perceptible vibration of the old pier by passing trains, the pumping was discontinued, as this operation tended to draw water through the old foundations, thus creating an element of danger.

To avoid the possibility of danger, it was decided to use an air-bell for completing the excavation and for depositing the concrete for the new pier. As the foundations for Piers *B*, *C*, and *D*, had been completed previous to the operations at Pier *N*, a compressor plant and a force of "sand hogs" were available. The inside dimensions of the coffer-dam were 17.5 ft. by 58.5 ft., the outside dimensions of the air-bell being 12 ft. by 48 ft., with a height of 5 ft. 0 in. This air-bell was constructed on Herr's Island, adjacent to the work, and was hoisted above the coffer-dam by a derrick boat, with three air-shaft sections attached. The bell was sunk to the bottom inside the coffer-dam, with a space of 2 ft. 6 in. between the sides of the air-bell and those of the coffer-dam. This space was filled with tremie-deposited concrete, the bell was completely covered, and its top and sides were sealed, after which the water

was pumped from the coffer-dam, and the concrete was deposited in the dry, to the water level.

Air pressure was then applied to the bell and the water inside was expelled. The underlying soil was removed, and as the rock was quite soft, excavation was continued 6 ft. into the sandstone. The concreting of the excavation, including the interior of the bell, was then performed under air pressure. The bell was of light construction, and the conditions under which it was used were distinct from those prevailing in the construction and use of ordinary pneumatic caissons. The absence of launching stresses or cutting-edge stresses, and the fact that the interior air pressure was resisted by the concrete enclosing the bell, conferred further advantages in this operation.

In the construction of the bell, 3 by 8-in., ship-lap pine sheathing was used, which was spiked to nine 6 by 10-in. pine cross-frames. The experience with the problems at Pier *N*, as described, indicates that the air-bell method as adopted was the most economical.

The plant of the Vang Construction Company at this point included a steel concreting tower, 225 ft. high, from which, after being mixed at a central plant, most of the concrete was deposited by chutes. Dinkey equipment was used to deliver the concrete to those points where it was impossible to deliver it by the chutes. A complete air-compressor plant was located at a site selected to meet the needs of the work. Although the work on the substructure for this bridge was carried on through a severe winter, which handicapped the operations of the contractors, an excellent record was maintained.

Erection Methods

Shipments of steelwork began to arrive at the bridge site early in May, 1920. The original intention was to use a sub-divided truss of 14 panels for the 436 ft. 9-in., through, truss span, *C D*, and 9 panels for the 265-ft. through, Pratt truss span, *BC*. The design finally developed for the 436 ft. 9-in. span was a simple triangular truss of 10 panels, having 20 panel points and 29 members per truss. The simple form of truss adopted eliminated secondary stresses to a great extent.

The work of erection began in June, 1920, and proceeded simultaneously from each end of the new steel structure. In scheduling this work, the contemplated grade change and the necessity for establishing the final grade at an elevation approximately 14 ft. above the old grade were considered. As stated previously, the line of the new structure diverges from that of the old in its westerly portion, near the Back Channel. The old Back Channel structure has been retained and adapted to single-track operation. This required the placing of a new floor in two of the old spans, the remainder of this structure consisting of the new single-track Herr's Island connection, crossing the Back Channel, which is designed for Cooper's E-50 loading.

As the new and the old alignment coincide at the east bank of Herr's Island, a temporary single-track detour was constructed over the Island on the up-stream side of the permanent line. The plate-girder spans which form the viaduct at the west end, from Abutment *U* to Pier *G*, were erected in final position. The through plate-girder span, *FG*, was erected 14 ft. south

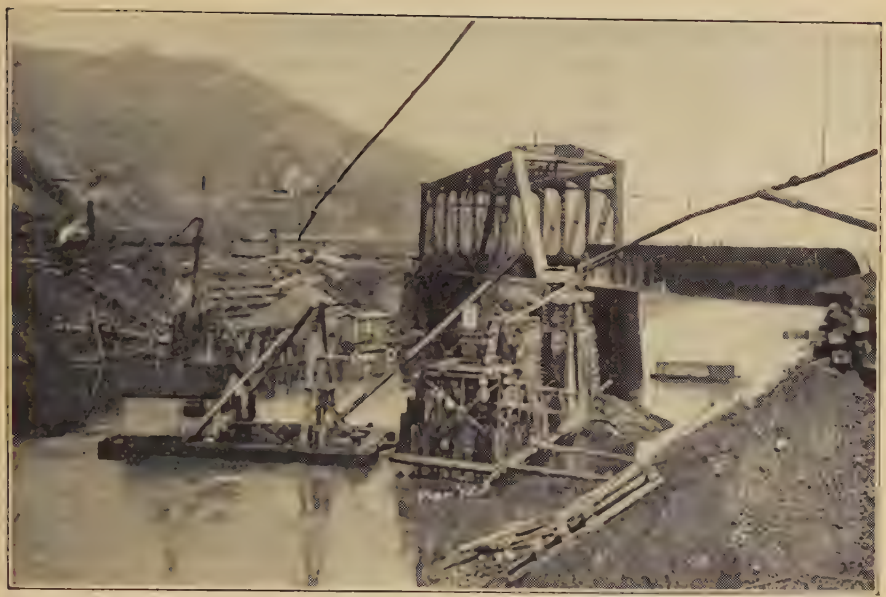
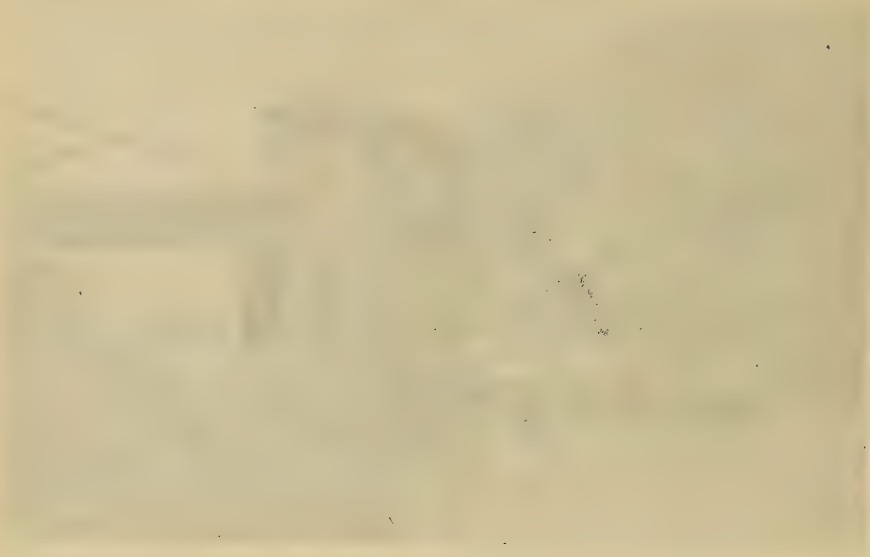
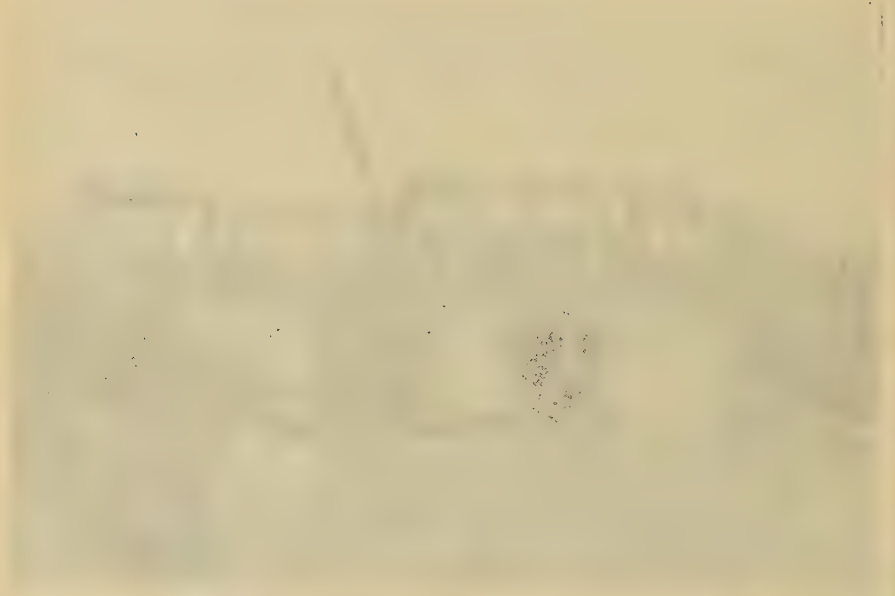


Fig. 2.—View of Pier *N*.

Fig. 3.—New Spans *AB* and *BC* in Place at Final Alignment.





of its final location, that is, its final west-bound track was used temporarily on the location of the east-bound track, this arrangement being made in order that the overhead clearance required for the connection track to Herr's Island could be secured, and in view of the necessity of obtaining adequate lateral clearances for the detour track. The east-bound track on Span *FG*, in this position, served the purpose of an erection siding, providing access to the 434-ft. span in its temporary position down stream from the old structure. In order to provide suitable lateral clearance for this track, it was further necessary to cut one corner of Pier *G*, and use a plumb face. The deck plate-girder spans, *DE* and *EF*, could not be erected in their final location, due to interference with the detour track.

At the east end of the structure, the 97-ft., deck plate-girder span, *AB*, and the 265-ft., through truss span, *BC*, were assembled and erected at the old or low level, on falsework extensions to the new piers (Fig. 3), built down stream from the bridge, falsework extensions to the old piers, for the support of the two old truss spans in their rolled-up position, being made on the up-stream side of the structure. During the erection of the 265-ft. truss span and the 97-ft. girder span, at the east end of the river structure, the river traffic passed under the third span from the east bank of the old bridge. After the 265-ft. through truss span had been moved to final line, the old pier beneath it was removed, and the channel dredged under the new span. The movement of river traffic was then shifted to this opening, and the falsework for the 436 ft. 9-in. channel span was placed. This method of disposing of the traffic on the river made it possible to erect both these spans on falsework, and obtain a completely riveted structure.

The 97-ft. girder span, *AB*, at the east end of this bridge, was supported in temporary position on double pile bents of 8 piles per bent, located down stream from the present structure. A connection was made between the tracks on this span and the east-bound track on the 33d Street Viaduct by a trestle and turn-outs. The west end of the 97-ft. girder span was fitted with special brackets which seated on the end floor-beams of the 265-ft., through truss span that carried the west end of the deck plate-girder span during the rolling operation.

For rolling, the east end of the 97-ft., deck plate-girder span, that is, at Pier *A*, was carried on two nests of rails of four lines each, with rollers between, 3 in. in diameter, spaced 8 in., center to center. The falsework for the 265-ft. span consisted of seven bents, one under each intermediate panel point, braced transversely with timber bracing and longitudinally at every other panel point by four lines of cross-bracing and by girts placed at the tops of the bents and also just above the water surface. Each bent consisted of nineteen piles, about 57 ft. long. I-beams were placed on top of the caps of the pile bents to distribute the panel loads.

To support the span during erection and during the rolling operation, the down-stream pier extensions, at Piers *B* and *C*, were made strong enough to permit the swinging of the span, thus supporting it as a simple span from end to end. At Piers *B* and *C*, additional castings 2 ft. 6 in. high were placed at the final location of the truss shoes, between which I-beams were placed, and bolted to the castings. I-beams were also placed from the end of the

pier to the falsework supporting the new span; the tops of the I-beams, together with the steel castings, furnished a continuous, smooth surface, on which the rolling took place.

The new span was moved on rollers, 3 in. in diameter, spaced 4 in., center to center. The power for the rolling operation of September 1st, 1920, was furnished by two locomotive cranes and a derrick car, distributed as follows: One locomotive crane at the ground elevation, near the up-stream end of Pier *A;* one locomotive crane at the east end of the 265-ft. span, at Pier *B;* and one derrick car at the west end of the 265-ft., through truss span, at Pier *C*.

The old spans were placed on rollers, 3 in. in diameter. These spans were blocked to the new spans, and were pushed out of place by the movement of the latter to final position.

Constant check of the movement of the spans during the rolling was maintained by transit observations on range boards. The cranes, derrick car, and the old and the new piers, were connected by telephone. The foreman and field engineer in charge of the operation were located at the telephone central, and thus had direct communication with all points of movement.

On September 1st, 1920, the 265-ft., through truss span, *BC,* and the 97-ft., deck plate-girder span, *AB,* were pulled to final line at the old grade (Fig. 3), and, simultaneously, two old truss spans, extending from old Piers Nos. 1 to 2 and 2 to 3, were withdrawn to a temporary position on the up-stream pier extensions. The total length of the old spans removed in this operation was about 412 ft., while the length of the new spans moved into place was about 360 ft., which left a gap of about 54 ft. between the new steel and the easterly end of the old span between Piers Nos. 3 and 4. This gap was bridged temporarily by a single-track, deck, plate-girder span, about 50 ft. in length, which was floated on barges to a position directly under the opening. After the movement of the two long spans, a derrick car was moved to the westerly end of the 265-ft. span, *BC,* and the 50-ft. girder span was raised from the barges and bolted into temporary position.

The chronology of the various operations incident to the performance of the work on September 1st, 1920, is summarized briefly, as follows:

10:00 A. M.—The track was broken.
10:10 A. M.—Started rolling the two spans.
10:27-10:30 A. M.—Spans reached final line, rolling ceased.
11:30 A. M.—Both ends of the truss span landed; rollers removed and track connection made at the east end, Pier *A*.
11:45 A. M.—Started to lift temporary girder span to be placed between new Pier *C* and the old pier in the middle of the river, this girder span having been provided with timber deck and rails, and placed on barges immediately below the opening.
12:00 M.—Temporary girder landed.
12:10 P. M.—Track in satisfactory condition at east end of bridge, work train removed derrick car from the structure.
1:03 P. M.—First train passed over the bridge.
1:20 P. M.—First passenger train passed over the bridge.

The total period of interruption to traffic was 3 hours 3 min. The period of 17 min. 30 sec. does not represent the actual time required for the movement of these spans, which was only 6 min. The difference, 11 min. 30 sec., was consumed by stopping the rolling operation three times, in order to check the position of the spans, and to make sure that all equipment was functioning in a satisfactory manner. The weight rolled was, as follows:

Span AB:	
Weight of new steel	318 000 lb.
Weight of deck	118 000 "
Span BC:	
Weight of new steel	2 098 000 "
Weight of deck	262 000 "
Approximate weight of equipment on span	670 000 "
Two Old Spans:	
Weight of metal	1 200 000 "
Weight of deck	310 000 "
Total weight	4 976 000 lb.

The old spans were then dismantled. The floor system and the top and bottom lateral bracing of old Span No. 2 were removed, after which the trusses were skidded to the new bridge, lashed to it, and dismantled. This procedure was impossible in dismantling old Span No. 1. After the floor system and laterals were removed, the trusses were lowered into the river, and then removed.

When this operation was completed, preparations were made for the placing of the 436 ft. 9-in., through truss span. At this stage of the work, all traffic was carried on a single-track line, on the west-bound side, from the crossing of the 33d Street Viaduct over the tracks of the Allegheny Valley Railroad, over the new Spans *AB* and *BC*, the temporary girder span, and the two old truss spans, to Herr's Island, thence over the detour trestle and the old Back Channel spans on the low level.

The remaining work may be divided into five parts, namely:

I.—The raising of the east-bound track on the 33d Street Viaduct, from the Allegheny Valley Railroad crossing to Pier *A*, a distance of 921 ft.

II.—The rolling into final position, at high level, of the 436 ft. 9-in., through truss span, *CD*, and the deck plate-girder span, *DE*.

III.—The rolling of the through plate-girder span, *FG*, a distance of 14 ft.

IV.—The placing of deck plate-girder span, *EF*.

V.—The jacking of the 265-ft. through truss span, *BC* to the new grade.

The raising of the east-bound track on the 33d Street Viaduct was started promptly. The stringers only were raised, and, where necessary, temporary steel pedestals and bents were provided between the bottom of the stringers

and the top of the cross-girders. This temporary steelwork was scrapped after both tracks of the viaduct had been raised and the cross-girders brought to their final elevation. The deck plate-girder span under the east-bound track in Span *AB* was also raised when the east-bound track on the 33d Street Viaduct was elevated. It was necessary, therefore, that Operations II, III, IV and V, be carried out at one time, when all traffic would be abandoned. The final change was made on December 20th, 1920, according to a plan each step of which was carefully arranged in advance.

The jacks and pulling tackle were tested the day before. The alignment of the moving spans was maintained by transit observations on horizontal gauge boards, and the foreman, transitmen, and engine operators, were provided with telephones.

The weight involved in the rolling operation of December 20th, 1920, was, approximately, as follows:

Truss Span CD:	
Steelwork	4 788 000 lb.
Timber deck	437 000 "
Equipment on span	720 000 "
Deck Plate-Girder Span DE:	
Steelwork	194 000 "
Timber deck	90 000 "
Two Old Truss Spans:	
Metal work	1 204 000 "
Timber deck	315 000 "
Total weight	7 748 000 lb.

The approximate distribution of this load at the respective bearing points was:

Pier *C*	2 972 000 lb.
Pier *D*	3 114 000 "
Pier *E*	143 000 "
Total	6 229 000 lb.

Prior to December 20th, 1920, the entire bridge was in operation at the low level, with only the west-bound track in service, as the east-bound track on the 33d Street Viaduct was being raised to take traffic at the new grade, after the final change of the river span.

At Piers *C* and *D* (Fig. 4), the rigging consisted of 20-part tackles, $\frac{3}{4}$-in., steel rope, and, at Pier *E*, an 8-part tackle and $1\frac{3}{4}$-in. manila rope were used. The roller nests consisted of rollers, 3 in. in diameter, 8 ft. in length, spaced 4 in. center to center, held by side-bars perforated to admit the ends of the rollers. Practice has shown that a load equal to the usual loading of expansion rollers is satisfactory, and data are available on the friction to be expected. The calculated roller load was 1 960 lb. per lin. in. of roller, or about 650 lb. per lin. in. of roller per inch of roller diameter. The rolling

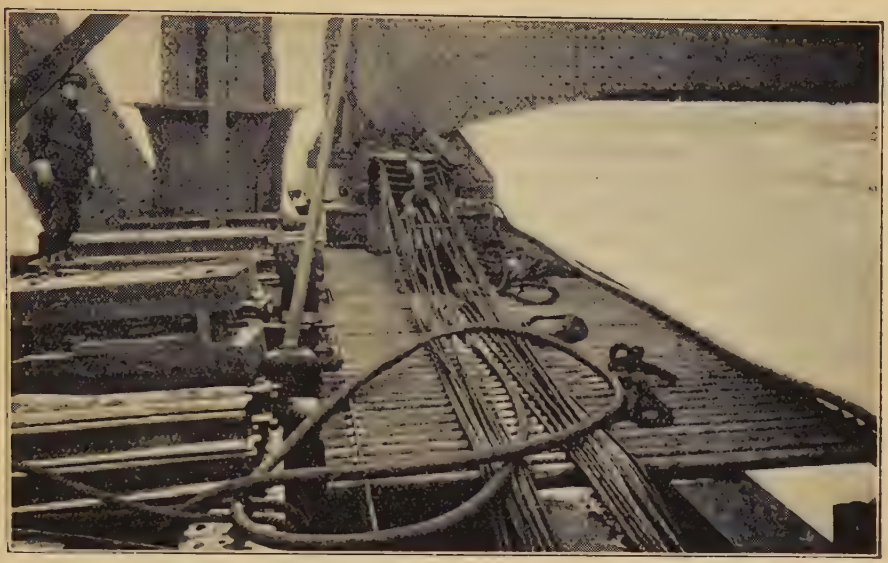
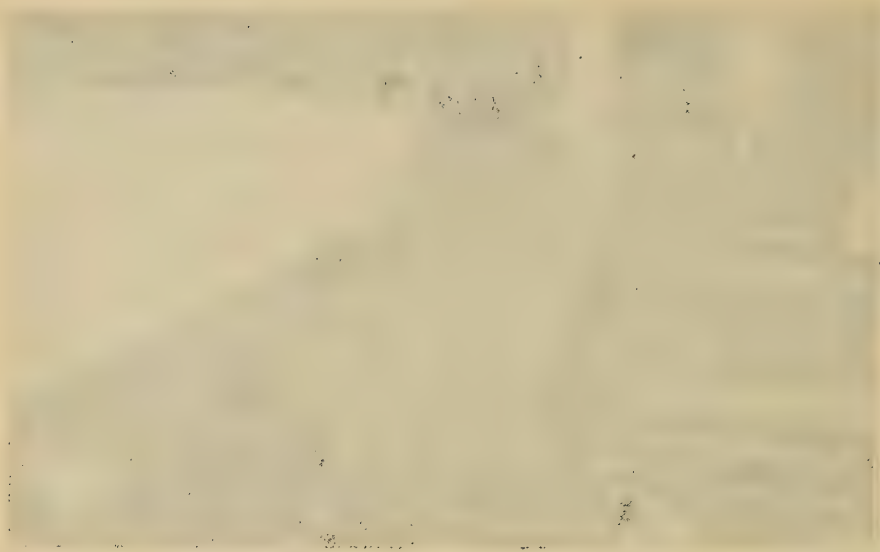


FIG. 4.—PULLING TACKLE AT PIER *C*.

FIG. 5.—OPERATING TRACK OVER OLD SPANS AT LOW LEVEL. NEW SPANS *CD* AND *DE* BEFORE ROLLING.



friction was assumed to be 0.006, based on a 1-ft. diameter of roller, or 0.024 for the roller 3 in. in diameter. Experience indicates that this amount is about 20% too high, when the track is supported so that no local crushing occurs.

The 436 ft. 9-in. channel span was erected at the elevation of the new structure; steel bents were provided to support it on the new piers after it had been rolled to final position on these piers which were about 14 ft. below their final height. As the shoe-plates of the new span were a little below the rail level of the old spans, it was possible to place the steel bent on Pier *C* without fouling the track, and this bent was moved in with the 265-ft. span. Pier *D* at the other end of the span, however, was located under the old structure, so that this bent could not be erected. It was built on falsework, as a part of the new span, with which it was moved into place.

The connection between the new and old spans was made by placing wooden blocking and struts between the posts of the two spans at a number of points. The force necessary to move the old spans thus had to be resisted by the lateral bracing of both the old and the new spans.

The falsework extensions provided for the erection and movement of the new span consisted of two 8-pile bents at each panel, each capped with one 12 by 12-in. timber and two 24-in. I-beams. At Pier *C*, the support consisted of pony bents framed on top of the timber pier extensions, which, capped at the low level, had served for the movement of the 265-ft. span. At Pier *D*, the new bent was framed to the full height. The track for the rolling consisted of five 30-in. girders.

Preliminary to the rolling, the old spans, the withdrawal of which was included in this operation, were raised by hydraulic jacks, four of which were placed under the end of each span. The jacks worked against short lifting girders, from which were suspended wire rope slings attached to the end pins of the respective spans. When the spans had been raised a sufficient distance, rollers were inserted, on which the spans were permitted to take bearing; the remainder of the roller nests were then introduced in the spaces between the shoes, and linked together. Rollers had been placed under the new span before it was swung. When the movement had been completed, the roller-nest segments between the shoes of the new span were disconnected and removed, after which the span was raised by jacks placed under the end floor-beams, to permit the removal of the rollers under the shoes.

The pier bent at Pier *D* was braced by struts extending from the base of the bent to the first panel point of the bottom chord of the span. At Pier *C*, the rockers of the expansion bearing, on which this span rested, were blocked by locking plates until the completion of the movement and the removal of the rollers.

The weather on Monday, December 20th, 1920, was ideal for the operation. After east-bound train No. 6 had passed, at 7 A. M., the track was cut. The temporary girders between Pier *C* and the east end of the old span were lifted out by the derrick car, and carried out of the way. The track was also cut at Pier *E*, to remove the I-beams carrying the detour or operating track across that pier. This left the pier clear for the rolling of the new spans, *CD*

and *DE*, and the withdrawal of old Spans 3 and 4. (Fig. 5.) Two locomotive cranes on the long truss span and a derrick car on the new west approach furnished the power for this operation. As soon as the new spans were in final position, they were jacked up to remove the rollers, and then lowered to permanent bearing on the masonry. The movement was accomplished in 14 min. actual moving time; and the total distance moved was 38 ft. 7⅞ in., making an average of nearly 3 ft. per min.

The chronology of operations in connection with the movement of the 436 ft. 9-in. through truss span and deck plate-girder span, *DE*, and the withdrawal of the two old truss spans replaced by them, is as follows:

7:00 A. M.—Track severed, immediately after the passage of east-bound train No. 6.

7:17 A. M.—Derrick car arrived at Pier *C*, for removal of temporary 50-ft. single-track deck plate-girder span.

7:40 A. M.—Locomotive crane arrived at Pier *E*, for the removal of parts of the detour trestle.

8:00 A. M.—Rolling of Spans *C D* and *D E* started.

8:54 A. M.—Rolling completed, total time 54 min.; actual time of rolling, 14 min.

10:15 A. M.—Span *D E* landed on Pier *E*.

10:23 A. M.—Span *C D* landed on Pier *C*.

11:30 A. M.—Span *C D* landed on Pier *D*.

12:00 M.—Span *D E* landed on Pier *D*.

As previously stated, the west-bound track deflected to the right, from the west end of the old river bridge, reaching a detour alongside the new west approach, which was maintained while this approach was being erected to final alignment. This viaduct was built complete to within three spans of the river bank, which spans had to be left until the final change, on account of interference with the detour.

While the work of removing the rollers from under the channel span was in progress, the steel bent on Pier *F* was extended to permit the rolling of through plate-girder span, *FG*, to final position. The rolling of this span started at 12:06 P. M., and was completed at 12:14 P. M., the total elapsed time of rolling being 5¼ min. Immediately after the placing of this span, the girders for Span *EF* were erected by derrick cars.

As soon as the 436 ft. 9-in. channel span had been moved to final position (Fig. 6), and the two old truss spans replaced by it had been withdrawn, the jacking of the 265-ft. through truss span to the new grade commenced, and was finished at 12:45 A. M., December 21st, 1920.

Jacking operations are difficult and complicated, whereas in the rolling of bridge spans there is little danger since breakage of the pulling tackle or failure of the power merely stops the movement.

Jacking is also a tedious operation; each lift is measured in inches, and the lifting is frequently stopped for re-setting. It is further essential that, as the load is raised, adequate supports must be provided to sustain it, which accounts for the great amount of time required for such work. Jacking operations

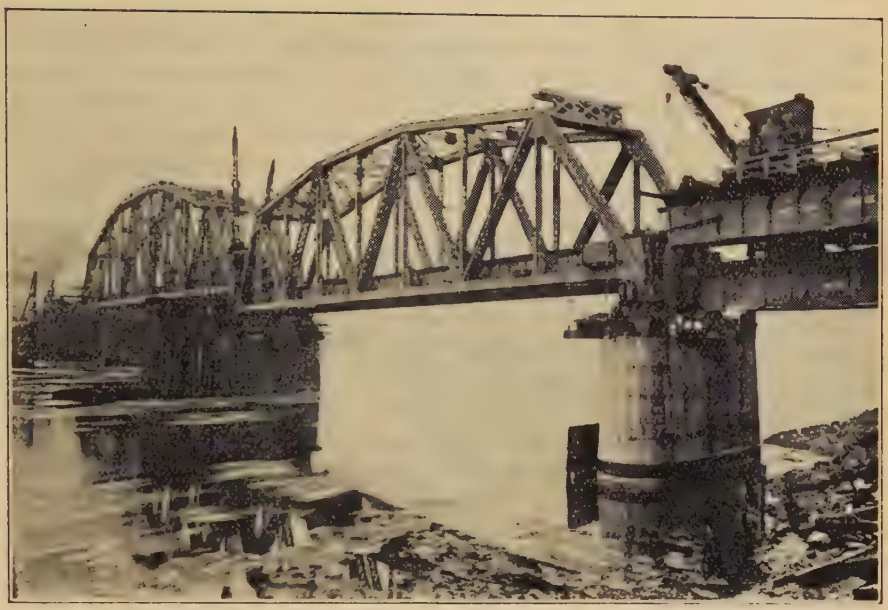
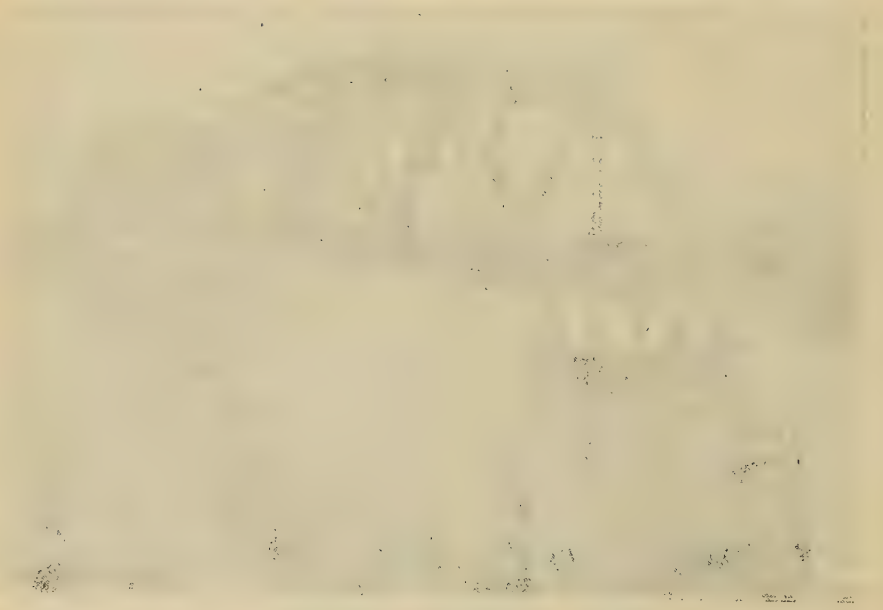


FIG. 6.—436 FT. 9-IN. SPAN AND ONE PLATE GIRDER SPAN IN FINAL POSITION. 265-FT. TRUSS PARTLY RAISED.

FIG. 7.—MAIN CHANNEL SPANS SUPPORTED ON STEEL BENTS.



involve an element of danger, and this procedure was adopted in the present instance only as a matter of necessity.

The 265-ft. through truss span was lifted a distance of 13 ft. 2⅝ in. at the east end, Pier *B*, and 15 ft. 2⅝ in. at the west end, Pier *C*. The load lifted was about 1 200 tons, or 600 tons at each pier. Four hydraulic jacks, each with a capacity of 500 tons, were used at each pier, in sets of two working and two idle. These jacks had a run-out of 26 in. By lifting the idle pair of jacks with the span, setting blocking under it, and then using these jacks for a further lift, a continuous lifting operation was possible. In order to avoid the carrying of the blocking to undue heights, the steel columns on which the span was to rest, were made in three sections.

To provide transverse bracing for the sectional columns, sectional girders or diaphragms were placed between the two columns as the raising progressed. These girders were used to support the jacks, and it was unnecessary, therefore, to build blocking for the jacks to the full height of the lift. After setting one story of the columns in place, it was necessary to lower the span on the columns, raise both jacks and remove the blocking under them, and insert the girder section between the columns. For the further operation, the jacks rested on the girder as previously they had rested on the pier. Two girders were provided at the west end of the span and three girders at the east end.

The lift was accomplished at each end in three successive raises of about 4 ft. each. A fourth lift was made at Pier *C*, for the purpose of inserting a cast pedestal intended to equalize the shoe heights of the two adjoining spans.

The piping was arranged so that at each pier the two spans were jacked as a unit. For the first fleet of jacks, both ends were carried up together; after that, the two ends were jacked separately. While this process was being carried on, the span was being followed up by rail grillages placed on top of the partly completed columns, arranged in such a manner that only one-half of this grillage was removed until the next column section was slipped half way into place. As a further safeguard, the heads of the jacks were followed by filler-plates surrounding the plungers.

All preparations on the two piers having been made in advance, the jacking started at 9 A. M., December 20th, 1920, and proceeded continuously, being completed shortly after midnight. The work of placing the girders between the pier columns proved slow, because of the skew of the girders with respect to their end connections and the columns. Work after dark also proved slow. (Fig. 7.)

Connecting up the new track required another hour after the span was entirely landed and secured. At 1:40 A. M., December 21st, 1920, the track was ready for traffic, and at 1:45 A. M., the first engine passed over the new structure, on the east-bound track, the only track in service at that time.

Before train service could be turned over the west-bound track, and double-track operation started, it was necessary to raise the girders for that track on the viaduct along 33d Street to the new grade. As already explained, provision for such a raise of grade was made when this viaduct was built; this was done by framing the first sixteen spans of longitudinal girders adjacent

to the bridge between cross-girders of the steel bents, while all the remaining girders of the viaduct were placed on top of the cross-girders. The raise of grade was accomplished, therefore, by shifting the longitudinal girders from between the cross-girders to the top flanges, supplemented by the use of shims or blocks of varying thickness; a further raise of grade in the seven spans nearest the river bridge was provided by designing the connections of the cross-girders to the columns so that the cross-girders could be moved a variable distance on the columns. As this last change could not be made until the girders for both tracks were raised, the girders for the east-bound track, which were raised first, were supported on temporary steel pedestals placed on top of the cross-girders.

During the time that traffic over the bridge was suspended, on December 20th, 1920, local passenger trains running west of Pittsburgh operated in and out of the Allegheny Passenger Station, while all through passenger trains were operated over the Pittsburgh and Lake Erie Railroad, between New Castle Junction and McKeesport.

With the completion of the major operation on December 20th, 1920, the concluding phases of the work were prosecuted with vigor, including the raising of the west-bound track on the 33d Street Viaduct to the new grade, already mentioned. On Thursday, January 27th, 1921, traffic into the 36th Street Yard was suspended at 7:30 A. M. At 2:40 P. M., Thursday, February 10th, 1921, the 36th Street Yard connection was put into service.

Traffic was likewise abandoned over the Herr's Island connection track crossing the Back Channel, while the floor system of this structure was being changed; train movement was resumed over this track at 3:45 P. M., Tuesday, March 15th, 1921.

At 7:00 A. M., Wednesday, March 16th, 1921, the west-bound track over the river bridge was thrown open to traffic.

Double-track operation was established over the river spans on March 16th, 1921, although this improvement was not formally placed in the hands of the Operating Department until Saturday, June 11th, 1921.

The approximate quantities involved in this work are as follows:

For the elimination of the Liberty Avenue grade crossing and the reconstruction of the 33d Street Viaduct:

Superstructure:	
Weight of new steelwork, in pounds	7 354 000
Timber deck, in feet, board measure	450 000
Concrete in solid floor spans, in cubic yards	459
Water-proofing in solid floor spans, in square yards	900
Substructure:	
Total cubic yards of concrete in piers, abutments, pedestals, and retaining walls	19 800
Total linear feet of concrete piles	43 200
Total cubic yards of excavation	13 000
Total cubic yards of fill	35 000
Total cost	$756 653 32

For the reconstruction of the Allegheny River Bridge, crossing Main Channel, Back Channel, Herr's Island, Herr's Island connection, and the 36th Street Yard connection, including the raising of the 33d Street Viaduct west of the Allegheny Valley Railroad Crossing, July, 1919, to June, 1921;

Raising 33d Street Viaduct:
Superstructure: new steelwork, in pounds. 396 305

265-ft. Truss Span *BC:*
New steelwork, in pounds.....2 098 000

436 ft. 9-in. Truss Span *C D:*
Superstructure: New steelwork, in pounds.4 788 000

Through Plate-Girder Span *FG:*
New steelwork, in pounds..... 412 850

Deck Truss Span *NO,* crossing Back Channel:
New steelwork, in pounds..... 849 284

36th Street Yard Connection:
New steelwork, in pounds..... 529 502

Single-track Herr's Island connection over Back Channel, including new floor system:
New steelwork, in pounds..... 686 814

Remainder of structure, consisting of deck plate girders between Pier *A* and the west abutment, *U:*
New steelwork, in pounds.....3 551 545

River and island structure:
Total weight of steelwork, in pounds.....13 312 300

New steel structure:
Total quantity of timber in floor system, in feet board measure..... 604 000

The substructure for the river and island spans contains the following approximate quantities:

Concrete, in cubic yards..... 26 100
Concrete piles, in linear feet..... 25 600

In the trestle approach west of Abutment *U,* and the trestle approach on the 36th Street Yard connection, the following approximate quantities are used:

Timber piles, in linear feet..... 24 000
Timber, in feet board measure..... 480 000

The fill at the west end of the structure, and the fill on the approach to the 36th Street Yard connection required a total of 84 000 cu. yd.

The cost of the elimination of the grade crossing at Liberty Avenue, the reconstruction of the 33d Street Viaduct, the river, Island and Back Channel structures, and other incidental work, amounted to approximately $3 000 000.

Organization

In the reconstruction of the 33d Street Viaduct, and the grade-crossing elimination work incident thereto, the interests of the Railroad Company were placed in the general charge of Francis Lee Stuart, M. Am. Soc. C. E., Chief Engineer, and of R. N. Begien, M. Am. Soc. C. E., who, on Mr. Stuart's resignation on July 10th, 1916, succeeded him as Chief Engineer. The work

on the superstructure, including the preparation of general and detail plans, fabrication of steelwork, and erection, was under the immediate supervision of Mr. W. S. Bouton, Engineer of Bridges. Paul Didier, M. Am. Soc. C. E., Principal Assistant Engineer, was responsible for the field work, and it may be mentioned, as a matter of interest, that Mr. Didier was also connected with the original bridge construction at this point in 1883-84.

In the reconstruction of the river and island structure, the Railroad Company's interests were placed in the general charge of H. A. Lane, M. Am. Soc. C. E., Chief Engineer. The superstructure work, including the preparation of general and detail plans, fabrication of the steelwork and erection in the field, was under the supervision of Mr. Bouton, until October 24th, 1919, and, from that date until the completion of the work, under that of the writer. The field work was under the supervision of Mr. A. C. Clarke, District Engineer, represented at the site by Mr. W. M. Ray, Assistant Engineer. The services of J. E. Greiner, M. Am. Soc. C. E., Consulting Engineer, were retained, and he was represented at the site by E. W. Stearns, M. Am. Soc. C. E. The American Bridge Company, placed in charge of the erection J. L. de Vou, M. Am. Soc. C. E., Division Erecting Manager, N. H. Orr, Assoc. M. Am. Soc. C. E., Assistant Division Erecting Manager, and Mr. Sherman Frantz, Superintendent.

DISCUSSION

EDWARD W. STEARNS,* M. AM. SOC. C. E. (by letter).—The writer was the local representative on this work for J. E. Greiner, M. Am. Soc. C. E., Consulting Engineer, during the construction of the river bridges and the westerly approach. Although Mr. Lang has discussed this work thoroughly and concisely, there are a few points which, by further emphasis, should make the problems more clear to one less familiar with the actual conditions. His brief treatment of the historical outline and of the problems overcome in the reconstruction of the 33d Street Viaduct in 1916 is sufficient.

In the treatment of the reconstruction of the river bridges and the westerly approach, and the further reconstruction of the 33d Street Viaduct, a direct statement or summary of the problems to be solved should have been given at the beginning. These are, in general:

(1) The construction of the main river spans and part of the Herr's Island girder approach on the same line and at a higher grade than that of the old bridge.

(2) The construction of the remainder of the westerly approach on a line distinct from the old line.

(3) The raising of that part of the 33d Street Viaduct which lay between Penn Avenue and the river bank and the construction of the new 36th Street Yard connection.

(4) The re-arrangement of the 36th Street Yard, the Allegheny River Line in the vicinity of the bridge, and the Willow Grove Yard, with the re-arrangement of the tracks of the line into Allegheny in the vicinity of the bridge.

(5) The accomplishment of all this work with practically no interruption to main-line traffic and a minimum of interruption to traffic in and out of the railroad yards and the stockyards on Herr's Island.

As much as this paper is primarily on the reconstruction problems of the bridge, no treatment is, very properly, accorded to the work done in the railroad yards. Nevertheless, this work was a part of the reconstruction program and is worthy of mention, although a description of it is unnecessary.

Although Mr. Lang mentions the construction of a timber trestle on Herr's Island, he does not make clear how important it was or to what extent it simplified a considerable part of the work. This trestle extended, on the east of the old line, from the old Back Channel bridge to the west end of the old main river bridge and so removed the traffic that it was possible to complete both the substructure and the superstructure from Pier *G* to the western end of the bridge without interference from or interruption to regular traffic. Furthermore, it was possible to complete the lower portions of Piers *E* and *F*, as was done with the main river piers, *B*, *C*, and *D*, prior to the main rolling and jacking operation on December 20th, 1920. Not only was the construction of this detour trestle one of the first things done on the work, but it was also arranged so as to be suitable, after the traffic had been turned

* Pittsburgh Representative, J. E. Greiner & Co., Pittsburgh, Pa.

into the new lines, for the final stockyards connection, with merely a lowering of the grade.

The author does not make clear just how all this traffic on the river spans and on the 33d Street Viaduct was taken care of during construction. Before work had been started, trains were operated over both tracks on the old bridge, a restriction being in force that no trains were allowed to pass on the river spans. When the detour trestle was complete, traffic was taken off the east-bound track on the river spans, and single-track operation maintained on the west-bound track. This provided a track from which to work on the substructures and on the construction of the new river spans, which were erected on falsework down stream from the old bridge. It also kept the 36th Street Yard connection in operation during the raising of the east-bound track of the 33d Street Viaduct, as this connection led off from the west-bound track. After the 436 ft. 9-in. span had been rolled into place and the 265-ft. span had been jacked up to the new grade, the traffic was immediately placed on the east-bound track. This facilitated considerably the removal of the old spans by placing a "blind" track on that side of the new bridge adjacent to the old spans in their rolled out position. It also made it possible to raise the west-bound track of the viaduct between the east end of the new river spans and the connection to the 36th Street Yard, prior to cutting the traffic off this connection. Furthermore, during this time a considerable portion of the new 36th Street Yard connection could be built without interfering with traffic, so that when finally traffic had to be stopped, the duration of the period of interruption was reduced to a minimum.

Although there have been more spectacular feats of erection, the writer believes there never has been a work of construction involving more varied phases to be accomplished within one day, than was accomplished on December 20th, 1920, on this work. Mr. Lang summarizes these under Items II to V, inclusive, page 1165, and again in his program on page 1170, but, in reality, there are a number of other phases which should have been mentioned or at least treated more fully.

One such point is the lowering of the grade of the detour trestle to conform to the new grade of the stockyards connection. About 200 men, in one day, placed this in such shape that traffic was resumed over it the next morning. Another point which should have been emphasized was the fact that, owing to the detour trestle passing across the end of Pier *F*, the steel bent upon which was rolled Span *F G*, could not be completed until traffic had been stopped on the detour trestle. Aside from tearing out the detour trestle, this bent had to be erected before the rolling operation could proceed.

Another point which should have been clarified was the reason that Span *F G* was rolled into place, after Span *D E*, whereas Span *E F*, just between them had to be erected in place. A glance at the plan of the bridge on Plate XIII reveals the fact that Piers *E* and *F* are at different skews, Pier *E* lining up with the river piers and Pier *F* with Pier *G*. With Spans *D E* and *F G* in the position in which they were erected prior to the rolling, it can be seen that there was no room in which to erect Span *E F*, until both spans had been rolled into their final position.

A few points in connection with the construction of the foundations are worthy of note. The concrete piles under some of the piers, referred to by Mr. Lang, were "Simplex" piles cast in place after a heavy steel shell had been driven to refusal, these shells being withdrawn immediately after they had been filled with concrete. The cut-off of most of these piles was 10 to 15 ft. below the ground surface, and yet, in few cases, after the excavation had subsequently been made, was it found that the top of the pile was far from the intended cut-off. Steel bars, to act as reinforcement, were introduced in the shells prior to pouring the concrete, but the writer believes their value as such is doubtful. Objection is heard at times to this type of pile in that the driving of adjacent piles is likely to disturb the set of the concrete in those piles which have just been driven or even to squeeze them off entirely. Regardless of the fact that these piles were driven approximately on 3 ft. centers and largely in a more or less loose cinder fill, subsequent excavation failed to reveal any indication of action detrimental to the set of the concrete, and only one case where there had been squeezing. This squeezing had been caused by an old timber which had been pushed out of the way by the shell.

On page 1158, Mr. Lang states that "the caissons were nosed into the shale, and excavation was continued for a distance of from 14 to 16 ft. below the cutting edge." Although not unknown, this condition is unusual and worthy of more detailed explanation.

When the cutting edge had penetrated the shale from 4 to 6 ft. and there was no indication of any better conditions being reached for a considerable distance, the caissons were "blocked up" and the shale removed for about 2 ft. below the cutting edge and well back beyond the outside faces of the caissons. This space was then filled with a rich, wet cement mortar, part of which was forced into the seams of the surrounding shale by the air pressure in the working chamber. This action not only helped to cement the loose shale into a more or less solid condition near the cutting edge, but also gave the caisson a much greater bearing area on the shale. The four heavy transverse struts across the inside of the working chamber were sufficient to withstand any tendency for the walls of the working chamber to collapse, due to a failure in the loose soft shale. A further precaution was taken by keeping the faces of the subsequent excavation well inside the inside walls of the working chamber. With these precautions, no extraordinary difficulties were experienced other than that before rock was encountered a point was reached at which the shale was so loose and porous that the air pressure would not exclude all the water, although the full capacity of the pumps was 3 700 cu. ft. of air per min. At no time, however, did the water enter in quantity sufficient to make it difficult to remove by blow-pipes. Blow-pipes were operated up to the time the concrete was poured into the excavation and working chamber, after which they were not used until the concrete had been placed well above the point where the air pressure would exclude all the water.

PHILIP GEORGE LANG, JR.,* M. AM. SOC. C. E. (by letter).—Work of this magnitude involves many complex and intricate details. An attempt to dis-

* Engr. of Bridges, B. & O. R. R., Baltimore, Md.

cuss all the secondary phases of the work would be unjustified, and the mass of accessory detail would probably disguise the importance of those features of general operation that might be of professional interest and merit treatment in a published article.

The intention of this paper was to describe concisely and intelligently the principal operations incident to the work, and the unique elements involved therein. All matters essential to a full comprehension of its salient features have been clearly set forth in the descriptive matter, or are apparent from the illustrations.

Although it is not believed that the phases mentioned by Mr. Stearns fall within the scope of the paper, the writer is gratified that these minor, but interesting, details have been preserved, and will find a place in the permanent records of the Society.

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Paper No. 1502

CORE STUDIES IN THE HYDRAULIC-FILL DAMS OF THE MIAMI CONSERVANCY DISTRICT

By Charles H. Paul,* M. Am. Soc. C. E.

With Discussion by Messrs. George L. Dillman, Allen Hazen, H. F. Dunham, Thomas H. Wiggin, H. S. R. McCurdy, Joel D. Justin, O. N. Floyd, J. Albert Holmes, D. C. Henny, A. L. Pauls, George L. Albert, and Charles H. Paul.

Synopsis

Much has been written from time to time about the condition and behavior of core material in hydraulic-fill dams. A systematic study of the subject has been carried on during the construction of the five dams of the Miami Conservancy District, with the result that many of the doubts that have existed heretofore regarding the stability of hydraulic-fill cores, have been dispelled. It will be shown in this paper that:

1.—The gradation of core material may be controlled during construction, even with borrow-pit materials of widely different character.

2.—A considerable excess of fines is required in the borrow-pit material, in order to maintain proper control of the core and to prevent the encroachment of gravel and sand into the core zone.

3.—A reasonably wide core, namely, core width at any point equal to height of dam above that point, may be obtained with absolute safety.

4.—A fairly high percentage of extremely fine material in the core is not objectionable, provided the material is properly graded.

5.—Such cores show a satisfactory rate of consolidation.

6.—Such cores are stable, after a few months of consolidation, to the extent that they will not flow when the support of the outside slope material is removed.

The five hydraulic-fill dams of the Miami Conservancy District form retarding basins to control flood discharges in the Miami Valley, Ohio. Table

* Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.

1 gives the maximum height, length, and volume of each of the Conservancy dams.

TABLE 1.—Size of Miami Conservancy Hydraulic-Fill Dams.

	Germantown Dam.	Englewood Dam.	Lockington Dam.	Taylorsville Dam.	Huffman Dam.
Maximum height, in feet.....	110	125	78	78	73
Length, in feet.....	1 200	4 700	6 400	3 000	3 300
Volume, in cubic yards.....	800 000	3 600 000	970 000	1 130 000	1 350 000

It will be noted that these dams vary in volume from 800 000 to 3 600 000 cu. yd., and in maximum height from 73 to 125 ft.

Fig. 1 shows the standard cross-section which was adopted for the Conservancy dams.

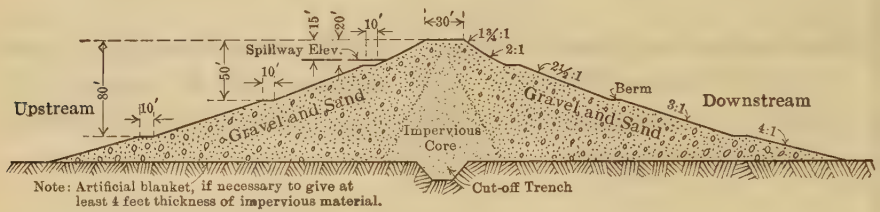

Fig. 1.

The construction of the dams was begun during the season of 1918, and all of them have now been completed. Two of them have already been in action during a flood flow in the river.

Borrow-Pit Studies Led to Hydraulic-Fill Construction

Before the type of dams to be built in the Miami Valley was decided on, careful examination was made of foundation conditions and of materials available for construction. The foundation conditions dictated the selection of earth dams at all the sites, and borrow-pit investigations indicated that hydraulic-fill dams would be practicable and economical. The decision as to hydraulic-fill construction was not made, however, before consultation with several experts in that type of construction, and after careful mechanical analyses of the borrow-pit material to determine the relative quantities of coarse and fine, and especially the percentage and character of the material that would pass the 100-mesh sieve. These analyses, as well as precipitation tests, indicated that both in quantity and quality the fines in the borrow-pit material would meet the requirements of a good hydraulic-fill core. Further tests by the United States Bureau of Soils, at which better facilities are available than in the ordinary laboratory, including comparisons with similar material from other hydraulic-fill dams, confirmed the preliminary conclusions, namely, that the embankment material available was well suited to hydraulic-fill construction.

Design of Core

Careful study was given to the design of the core for the Conservancy dams, with the result that the core width at any point was made equal to the height of the dam above that point. This gave a core of such shape that sloughing in of the outside slope material would not be encouraged, of sufficient width so that, with reasonable care, the encroaching of sand and gravel too far into the core zone could be prevented, and narrow enough so that additional material to that required to give stable slopes would not be necessary in order to hold the pressure of the semi-liquid part of the core during construction.

In order to obtain cores of the dimensions decided on, it became necessary, at one of the dams, to waste a considerable quantity of the core material available in the borrow-pit, and, at two others, to open auxiliary pits in clay deposits on adjacent hillsides, to make up for deficiency of fines in the main borrow-pits. There are decided advantages in having a surplus of core material available, so that a part at least of the finest of that material may be wasted: First, a better gradation of core material may thus be obtained; in fact, after the selection of the borrow-pits, and such selection of material in the pit as is practicable as excavation proceeds, the wasting of fines affords about the only remaining opportunity to control the gradation of the core material; and, second, a more rapid consolidation of core results, by getting rid of some of the extremely fine particles which, otherwise, are held in suspension for a long period, tend to lubricate the whole mass, and retard the process of consolidation by the holding up of particles which of themselves would settle more rapidly.

The Goldbeck Pressure Cell

In order to study the action of the hydraulic-fill cores during construction, it was determined to place at various elevations, in all the dams, a number of Goldbeck pressure cells. Preliminary experiments with these cells by the U. S. Bureau of Public Roads, at the writer's suggestion, indicated that they would give reasonably correct results if placed in hydraulic-fill cores. A full description of this apparatus, as well as a discussion of the experiments referred to, have been published.* Briefly, the Goldbeck cell (Fig. 2) is a closed, flat, circular box, 5½ in. in diameter, similar in shape to a shoe-blacking box, the top or bottom of which acts as a movable diaphragm. It is operated by slowly admitting compressed air, through a small pipe, to the inside of the box. The air is supplied from a small tank which may be charged with an ordinary tire pump. The cell is buried at a known elevation in the core, and the movable diaphragm is held down by the pressure of the material. When the pressure of the air on the inside equals the pressure of the material on the outside, the diaphragm is lifted slightly, an electric contact is broken, and an indicator light in the registering apparatus goes out. At that instant, the gauge on the air line indicates the pressure of the material against the movable diaphragm. The operation of the cell requires a movement of the diaphragm of less than 0.001 in. The cell may be set either horizontally or vertically, and, thus, either vertical or lateral pressure may be determined.

* *Engineering News-Record*, April 18th, 1918, p. 758.

In addition to the Goldbeck cells, it was planned also to use the ordinary rod and ball tests, as construction proceeded.

Construction Methods

The embankment material was pumped into each of the Conservancy dams, by 15-in. dredge pumps, as it was impossible to sluice directly from the borrow-pit to the embankment. In two cases, however, the borrow-pit material was broken up by hydraulic giants and sluiced to sumps where it was picked up by the dredge pumps and pumped into place. At the other three dams, the borrow-pit material was excavated by drag-line machines, loaded into cars, hauled to the dams, and dumped into long shallow bins or hog boxes. At the hog boxes, hydraulic giants were used to wash the material to the dredge pumps. Thus, it will be seen that, by the time it reached its place in the dam, the borrow-pit material was thoroughly broken up and separated.

The discharge pipes laid down this material along the outer edge on either side of the dam section; a low levee or dike, along the outside slope, forced the discharge to flow toward the center where the core pool was maintained. The usual separation, characteristic of hydraulic-fill construction, was thus obtained. The coarser material, consisting of cobbles, gravel, and coarse sand, remained in the outer parts of the embankment, and the fine sand, clay, and silt, flowed to the center with the water, and settled through the core pool, to form the impervious core.

Distribution of Material in Dam Section

In every case, this general separation of material was obtained to the extent that the outside slope material was pervious, free draining, and composed largely of coarse particles, whereas the cores were impervious and of fine material. It should be noted, however, that the theoretical gradation of material outside the core zone, from coarse at the outside to fine toward the center, is hardly perceptible, except by careful examination, and then only in a general way. Although the material near the outside slope line contains a larger percentage of coarse particles and is more pervious than that near the edge of the core, yet considerable coarse gravel is distributed through the section of the dam from the outside slope to the edge of the core, and there is considerable sand near the outside slope as well as near the core zone, which is of no importance. It is also true, however, that a considerable quantity of the fines is trapped in this outside slope material, which is of importance and must be taken into consideration in any analysis of borrow-pit material. Unless such analysis shows a large surplus of fines in addition to that required by the theoretical core section, it is almost certain that the necessity of an additional supply of core material will develop during construction. This happened at two of the Conservancy dams, and although it was not entirely unexpected at one of them, it had not been anticipated at the other.

At all five of the dams there was a distinct deposition of sand at the edge of the core pool, forming a definite separation between the slope ma-

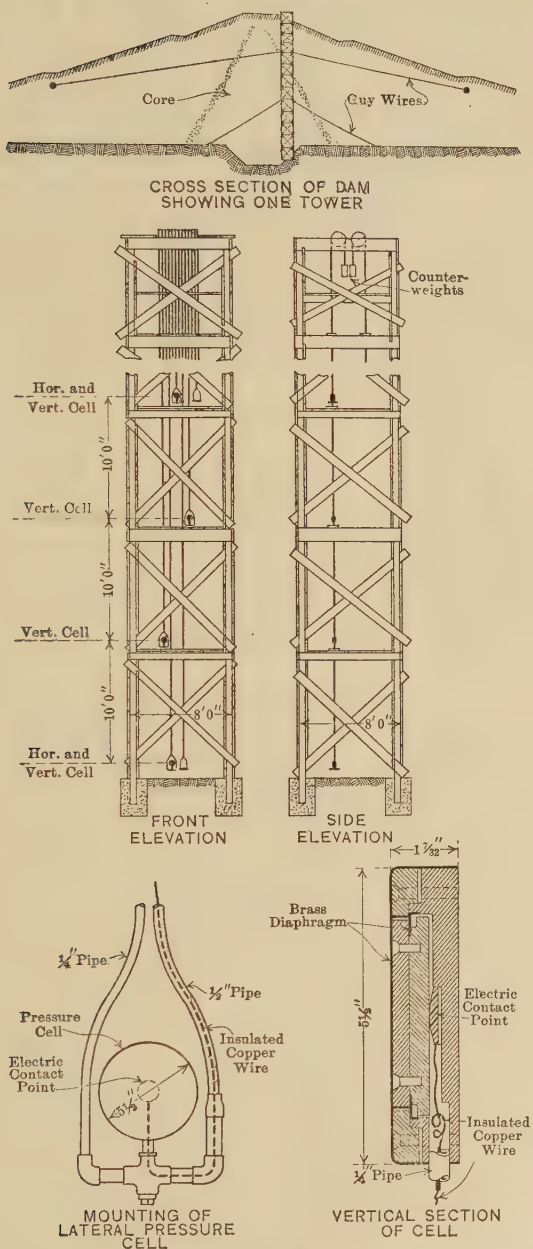
Core
Guy Wires
CROSS SECTION OF DAM SHOWING ONE TOWER
Counter-weights
Hor. and Vert. Cell
Vert. Cell
Vert. Cell
Hor. and Vert. Cell
10'0"
10'0"
10'0"
8'0"
8'0"
FRONT ELEVATION
SIDE ELEVATION
1 1/32"
Brass Diaphragm
1/4" Pipe
1/2" Pipe
Pressure Cell
Insulated Copper Wire
Electric Contact Point
6 5/8"
Electric Contact Point
Insulated Copper Wire
1/4" Pipe
MOUNTING OF LATERAL PRESSURE CELL
VERTICAL SECTION OF CELL

FIG. 2.

terial and the core material. The water flowing down the beaches from the discharge end of the dredge pipe, dropping the gravel and coarser material along the way, carried still an appreciable quantity of coarse sand when it reached the core pool. Its velocity was checked as it entered the pool, and its burden of coarse sand was immediately dropped, only the finest of the material, that is, the fine sand, clay, and silt, going into the core zone where it settled slowly through the core pool. This, in general, was what happened when construction conditions were right. Irregularity in shore line, concentration of flow down the beach, deficiency of core material, and similar unfavorable conditions which continually occur on construction work, regardless of how well it is organized, occasionally resulted in tongues of the coarser material extending into the core zone, thus emphasizing the necessity of a reasonably wide core section to take care of such contingencies which, however undesirable, can hardly be eliminated. The matter of gravel and sand sloughing or raveling into the core will be discussed in detail later.

Core Tests

The limitation of fines in the core material of the Conservancy dams was largely a matter of judgment, based on the analysis of core materials available, preliminary tests made in co-operation with the U. S. Bureau of Public Roads, and analyses of cores secured from other hydraulic-fill dams. After construction was started, frequent analyses of the core materials were obtained through the co-operation of the U. S. Bureau of Soils, and the rates of consolidation were studied. Table 2 shows the results of some of the analyses of typical core material.

Although cores of quite different compositions from those given may show as good results, it may be said that all five of these dams have been completed without the least indication of trouble with any of the cores, with ample proof that consolidation has taken place in a thoroughly satisfactory manner, and that the water-tightness of the core, as shown by several tests, is all that could be desired. In the writer's opinion, the most striking fact revealed by a study of Table 2 is that, with proper care, cores remarkably similar in composition can be obtained from different borrow-pit materials, in different river valleys, at points from 10 to 50 miles apart. It is not claimed that the gradation shown by these analysis cannot be improved, but the results obtained at the Conservancy dams indicate conclusively that cores of about the composition shown are eminently satisfactory from the standpoints of rate of consolidation, degree of consolidation, and water-tightness. To call to mind, again, the different conditions under which these cores were obtained, it may be stated that at Lockington and Englewood the borrow-pit material was used as it came, with only such control as could be secured by varying the depth of excavation, thus obtaining a greater or less proportion of the clay over-burden, or by such other selection of materials as could reasonably be made without unduly increasing the cost of excavation. At Germantown and Huffman, the main borrow-pits were deficient in fines, and auxiliary borrow-pits, yielding a high percentage of clay, had to be opened. At Taylorsville, there was a large surplus of fines in the borrow-pit, and a

TABLE 2.—Mechanical Analyses of Typical Core Samples.

Bureau of Soils, standard sizing.	Germantown.			Englewood.					Lockington.				Taylorsville.						Huffman.				Average.
Percentage of fine gravel, 2 to 1 mm	0	0	0	0	0	0	0	0	0.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Percentage of coarse sand, 1 mm to 0.05 mm	0	0	0	0.4	0	0	0	0	0.9	0	0	0	0	0	0	0	0	0.1	0	0	1.1	0.7	0.1
Percentage of medium sand, 0.5 to 0.25 mm.....	0.1	0	0	0.3	0.2	0	0	0.5	0.7	0	0	0.5	0.1	0	0	0	0	0.2	0	1.8	0.8	0.8	0.3
Percentage of fine sand, 0.25 to 0.10 mm	1.5	4.2	2.1	4.6	1.6	3.6	0.5	2.6	8.6	1.0	1.5	3.1	1.9	0.5	2.0	0.4	0.1	3.5	2.8	18.7	2.9	6.2	3.0
Percentage of very fine sand, 0.10 to 0.05 mm.	14.0	17.3	21.2	19.6	16.8	19.3	12.4	5.4	12.6	10.1	33.2	39.0	18.1	19.7	13.9	2.3	4.2	24.6	15.0	25.4	31.4	23.0	18.2
Percentage of silt, 0.05 to 0.005 mm	64.2	56.0	55.2	52.7	57.7	55.2	72.0	60.5	52.6	59.3	55.8	40.4	59.7	60.8	61.8	62.9	62.1	59.2	52.5	46.2	50.2	49.2	55.9
Percentage of clay, 0.005 to 0 mm	20.3	22.5	21.5	22.6	23.9	22.6	15.2	31.0	24.8	29.3	9.5	16.9	20.2	19.1	22.3	34.3	33.7	21.4	29.8	12.9	13.7	20.4	22.2

considerable percentage of the core material had to be wasted. In the latter case, there was the ever-present temptation to decrease the cost by holding a large percentage of fines, and taking chances on a wider core than that called for by the design. In the other case, one had to reckon with the desire to narrow the core to fit the borrow-pit conditions. Perhaps both these modifications might have been made, within reasonable limits, without detriment to the work. In the light of present knowledge, however, it is believed that either would have been undesirable, and might have led to trouble.

Pressure Cell Tests.—About fifty of the Goldbeck pressure cells were placed in the cores of these five dams. Some have given satisfactory readings, and some have not. In cases where the readings seem unreasonable, or where the cells fail to register, it is sometimes difficult to determine whether the trouble is in the cell or is due to faulty installation. With a few exceptions, there is no evidence that any of the unsatisfactory results have been due to any fault in the cell itself. Defective installation is known to have been the cause of many of the unsatisfactory readings, and improvements in installation have resulted in marked improvements in results. This was a pioneer effort, and, naturally, much was to be learned by experience, before proper methods were determined for meeting the requirements, particularly as to the best methods of installation. The first installation was made at the Germantown Dam, as shown in Fig. 2. At this dam, the cells were suspended, by two ¼-in. pipes, from the top of a framed tower which was built in sections as the construction proceeded. Two pipes were necessary at these installations, one to carry the wire connecting with the registering apparatus and to take the charge of compressed air, and the other to be used as a blow-off in case water entered the pipe or the cell. This method of installation, however, was improved later. (Fig. 5.)

Cells were set in a vertical position (to measure lateral pressures) at 10-ft. vertical intervals, and in horizontal position (to measure vertical pressures) at 30-ft. vertical intervals. As the tower was raised in height, the pipes were extended by connecting another section at the top, and another length of wire (bell cord) was spliced on and threaded through the one pipe. The tower, surrounded by the soft core, was guyed at intervals to hold it in place.

In taking the readings, a portable gauge box, Fig. 3, was taken out to the tower, and connected to one of the pipes from which the cell to be tested was suspended, the pipe being connected to the pressure tank and the wire to the indicator light. A plug was removed from the end of the other pipe and a light "shot" of compressed air was admitted to the first pipe to displace any water which might have accumulated. It was only necessary to force this water out of the way so as to admit air to the cell, since the weight of a water column (if any) in the second pipe would register on the pressure gauge, and, therefore, would not affect the correctness of the reading; as a matter of fact, little trouble with water was encountered. The plug was then replaced in the end of the second pipe, and the compressed air was admitted slowly to the cell, until the movement of the diaphragm broke contact and put out the light in the gauge box. At that instant, the pressure gauge indicated the core pressure on the diaphragm. The movement of the diaphragm necessary

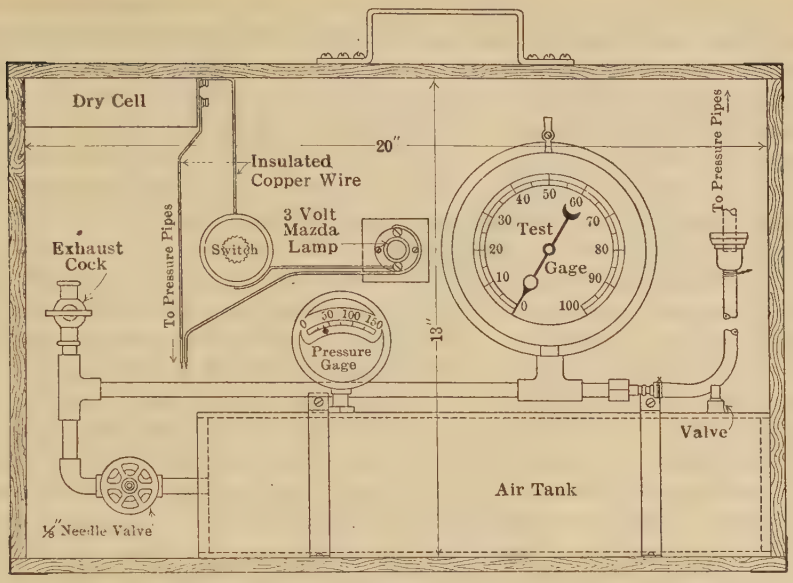

FIG. 3.

to break contact (about 0.0001 in.) is so slight that the light may be put out and lighted repeatedly, by proper manipulation of the feed and exhaust valves, without changing the reading of the pressure gauge, thus showing that the slight movement of the diaphragm does not disturb the core.

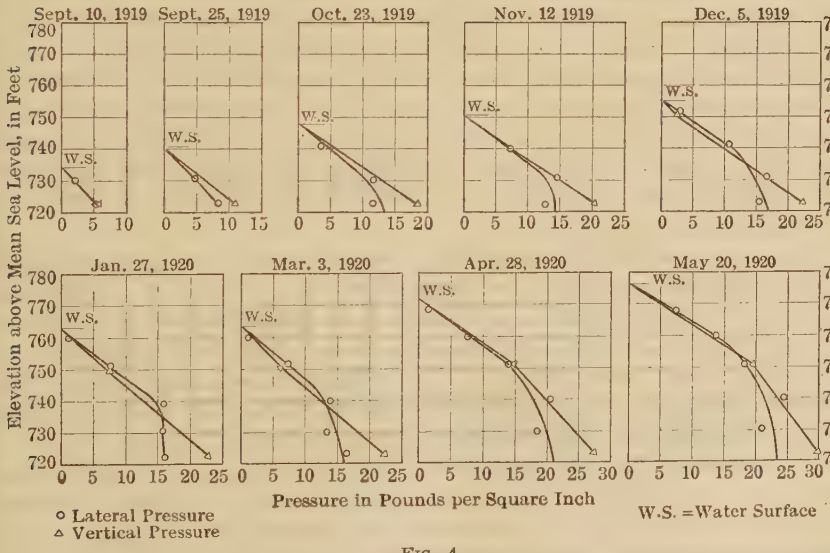

FIG. 4.

A series of readings at Germantown, covering the period from September 10th, 1919, to May 20th, 1920, are shown on Fig. 4.

These tests show that after a few weeks the core material begins to consolidate (as indicated by the fact that the lateral pressures are noticeably less than the vertical pressures at the same elevations) and that as time goes on, the difference becomes greater, until after a few months the lateral pressures in the lower parts of the core are only about two-thirds to one-half the vertical pressures. In the writer's opinion, this is conclusive evidence that the core at those points is losing its fluid properties and that consolidation is taking place. These conclusions are confirmed by later examinations of exposed cores, as will be mentioned.

These nine series of readings at Germantown, covering a period of eight months, were very satisfactory. Then things began to happen. The dam had reached a height of from 60 to 70 ft. and was being constructed at the rate of about 6 or 8 ft. per month. It became impossible to obtain further readings on some of the lower cells because air could no longer be forced through the pipe to the cell. This plugging of the pipe was due, in some cases, to the bunching of the insulation on the wire or at the splices, and, in others, to an accumulated deposit of dirt and flakes of galvanizing. When the length of this small pipe is considered (70 to 80 ft. in the case of the lower cells), this latter trouble is not surprising, although it was overlooked at the start. The use of larger pipe, more careful cleaning of the inside of the pipe, and the construction of a trap at the bottom of the pipe to catch the dirt, corrected these troubles in later installations. Some of the pipe joints then began to leak air, and, about that time, the upper part of the tower showed signs of distress, due probably to movement in the upper (partly fluid) portion of the core or to the settlement of material on the guys. This movement of the partly consolidated portion of the core was demonstrated by sinking a cast-iron ball into it, marking its position by a buoy, and checking that position from time to time. This simple test gave positive evidence that, in the earlier stages of consolidation, there is often some movement, or surging, of the semi-fluid core. Such movement, in itself, is of little consequence, except that it must be given consideration in cases where it is desired to construct a permanent tower or shaft within the limits of the core. Difficulties with such towers or shafts have been experienced in other hydraulic-fill dams, due no doubt to this core movement.

Thus, the pressure-cell tests at Germantown, following the period covered by the readings shown in Fig. 4, were of interest, principally in revealing defects to be corrected in future installations. The Englewood and Lockington installations were too far along to be corrected by the time the Germantown troubles had been analyzed. The readings obtained at these places confirmed the conclusions reached from the Germantown tests, but little additional information was secured, beyond what is shown in Fig. 4.

At Taylorsville, full advantage was taken of the experience gained at the other dams, and a much better installation was made. In this case, the cells were fastened to the side of a concrete cut-off, on the back of a high retaining wall forming a part of the outlet structure. Larger pipes were used, which were fastened securely to the same wall, and instead of joining up the pipe in sections as the dam was raised, the complete installation was made at the

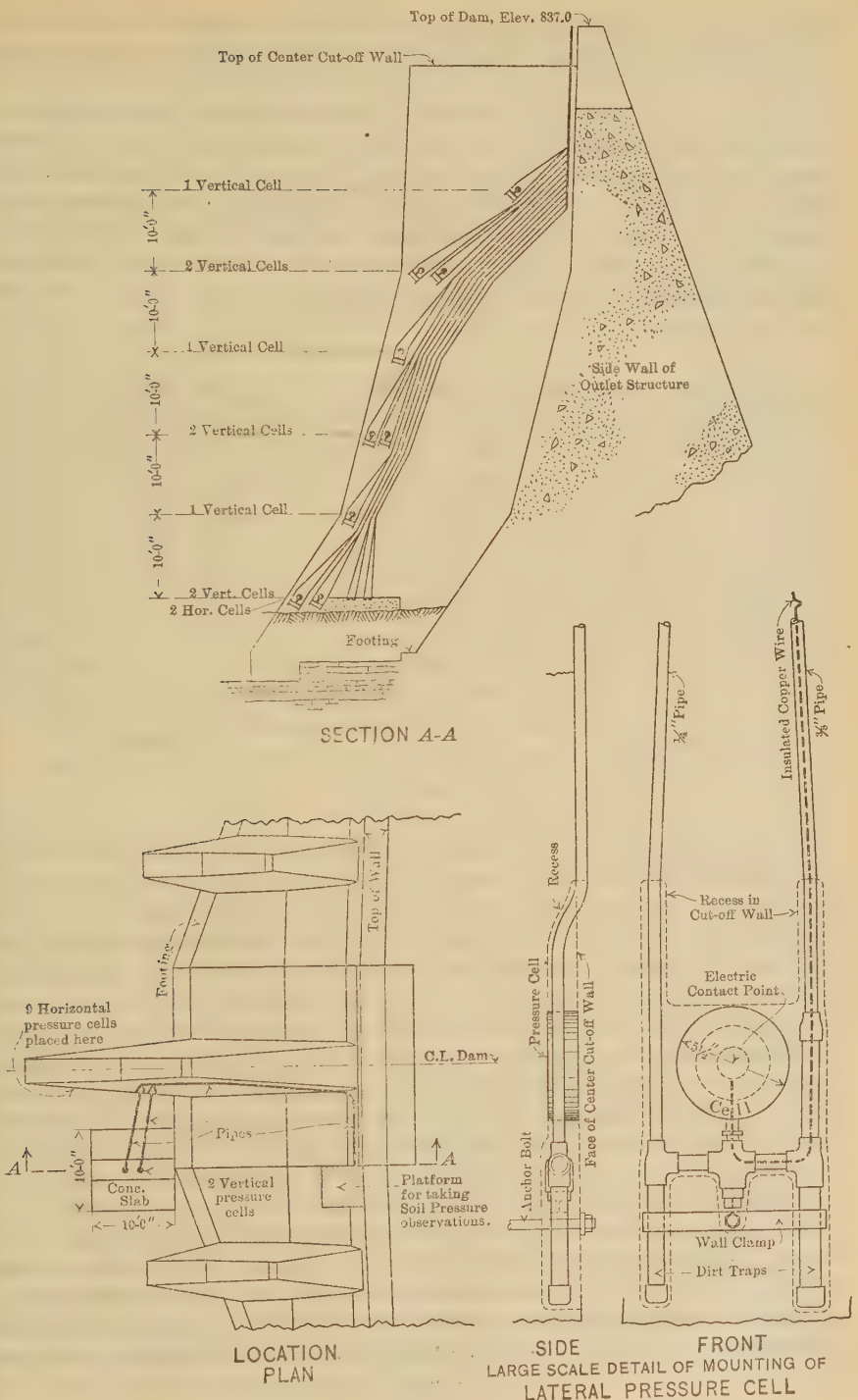
Top of Dam, Elev. 837.0
Top of Center Cut-off Wall
1 Vertical Cell
2 Vertical Cells
1 Vertical Cell
2 Vertical Cells
1 Vertical Cell
2 Vert. Cells
2 Hor. Cells
10'-0"
Footing
Side Wall of Outlet Structure
SECTION A-A
Top of Wall
Footing
9 Horizontal pressure cells placed here
C.L. Dam
Pipes
Conc. Slab
2 Vertical pressure cells
Platform for taking Soil Pressure observations.
LOCATION PLAN
Recess
Pressure Cell
Face of Center Cut-off Wall
Anchor Bolt
SIDE
¾" Pipe
Insulated Copper Wire
Recess in Cut-off Wall
Electric Contact Point
Cell
Wall Clamp
Dirt Traps
FRONT
LARGE SCALE DETAIL OF MOUNTING OF LATERAL PRESSURE CELL

FIG. 5.

start. At the pipe joints, special care was taken to secure a permanently air-tight joint. Dirt traps were provided at the bottom of all the pipes. (Fig. 5.)

Cells in vertical position, to measure lateral pressures, were set at 10-ft. vertical intervals as before, but in this case two cells were set at the bottom and at the 20-ft. intervals, in order to check the readings of one against the other, at those points. Cells to measure the vertical pressures were set at the bottom only, because of the difficulties of making a secure installation in horizontal position at the higher elevations, except at points where the vertical pressure would be affected by friction against the wall. Fig. 6 gives the results of readings at Taylorsville.

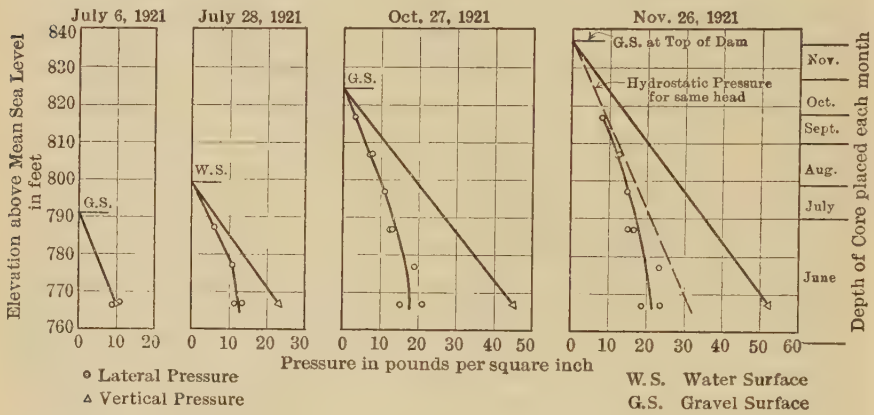

Fig. 6.

It will be noted that the two cells at the 20-ft., vertical intervals check each other very closely, except at the bottom, where consolidation has taken place to the extent that the lateral pressures are only about 40% of the vertical pressures. This is an indication that the pressure-cell readings are fairly reliable. It might be expected that when consolidation has reached the point where the core is practically a solid, actual lateral pressure might vary at different points to the extent indicated by the readings on the lower cells. The last set of readings (November 26th, 1921) were taken after the completion of the dam to its full height. The section of the dam in which the cells were set, was built up from Elevation 767, where the lowest cells are located, to Elevation 837, the top of the dam, during a period of 6 months; during 1 month the core was raised 33 ft. The monthly progress of construction of that section is shown on Fig. 6.

The Taylorsville method of installation is apparently satisfactory. The results to date have every indication of being reliable; they check with the readings at Germantown, as shown in Fig. 4, and they appear reasonable in the light of examinations of the cores in place, discussion of which will follow. The readings at Taylorsville will be continued for several months, or as long as any further information can be secured from them.

It is evident, from the data secured thus far, that properly graded core material need not be treated as a perfect fluid even after a few weeks of settlement, and that after it has been in place for a few months, the lateral pressure is only about 50% or less, of the vertical pressure at the same depth. It is believed that this may be accepted as a fact, to be kept in mind in the future design of hydraulic-fill dams.

Ball and Rod Tests.—Penetration tests with a 6-in. cast-iron ball, and with iron rods, were made at all the dams as the work progressed. These tests have been already described,* and as they are not of great importance, the discussion will not be repeated here. They indicated satisfactory and progressive consolidation of the core in all cases. Rod tests are of value in detecting the presence of gravel in the core.

Actual Core Examinations.—By far the most satisfactory and conclusive information as to the character of the cores of the Conservancy dams, has been obtained at three of the dams, by examination of the cores where they have been exposed in their undisturbed condition. At the Huffman Dam, the old line of the Erie Railroad passed through the dam site. The construction of the dam was started while the railroad was being re-located and in order not to delay this construction, a cross-dam was built along the edge of the railroad right of way, which held back the hydraulic core until after the tracks were moved. The core pool was then drained, the preparation of the foundation was made across the railroad right of way, and the cross-dam through the core section was removed, exposing the end of the core to a depth of 26 ft. Fig. 7 shows the core thus exposed.

Although 16 ft. of water stood against the exposed face of the core, and although it stood unsupported to a height of about 10 ft. above the water surface, little difficulty was experienced in cleaning the foundation for the new section of the dam and excavating the continuation of the cut-off trench. The texture of the core in the steep face, and how the bucket load of core, dumped by the drag-line machine, retains its shape, may be noted in Fig. 7, The oldest of the core visible in this photograph had been in place about five months, and the top of it had been placed about two weeks before the photograph was taken. It would bear the weight of a man walking across it, after the core pool had been drained. The composition of this core material is represented by the Huffman analysis in Table 2. When studying this photograph, it should be kept in mind that the face of the core, below the water surface, was undercut by the excavation for the removal of the cross-dam and the continuation of the cut-off trench, which explains the slumping near the face.

At the Lockington Dam, when construction had been carried to within about 12 ft. of the top, the core pool overflowed, washing a gully in the up-stream slope of the dam and exposing the core to a depth of about 30 ft. from the top. A similar accident occurred at Englewood, at about the same stage of completion.† In each case, all the water in the core pool drained through the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 1797, 1799 and 1800.

† *Engineering News-Record*, November 3d, 1921, p. 744.

break. At Englewood, the quantity of water drained was estimated at not less than 800 000 gal. In both cases, the core stood up with almost vertical faces and none of it was washed out, except just at the break. Only about 10% of the material moved by the wash-out was core material. Figs. 8 and 9 show the conditions at these dams immediately after the wash-outs occurred.

It will be noticed in both photographs that the core stands up, unsupported, in almost vertical faces, and that there was no slumping of the core, except just at the wash-out. It is significant also, that there was little or no erosion of the core back from the break on either side, although a large quantity of water flowed over it when the wash-out occurred. (Fig. 10.) Nor was there any appreciable sloughing of the core during the three or four weeks that it stood unsupported while the wash-outs were being repaired. At Englewood (Fig. 9), the top of the core had been in place for less than two weeks before the wash-out. The oldest of the core in sight in the photograph had been in place only about twelve weeks.

Samples of the core material from these places were taken in both cases. At Lockington, five samples were taken at different places, after digging into the exposed face far enough to reach material that had not been disturbed or affected by the wash-out. Each of the samples was secured by using a 7-in. length of stove pipe, biscuit-cutter fashion, and immediately soldering on a top and a bottom, thus forming an hermetically sealed can, full of core as it lay in the dam. The moisture content of these five samples, as determined by the Bureau of Soils, varied from 21.8 to 22.6% by weight. The mechanical analysis averaged, as follows:

1-0.5 mm.	0.5-0.25 mm.	0.25-0.10 mm.	0.10-0.05 mm.	0.05-0.005 mm.	0.005-0 mm.
0	2%	6%	15%	55%	21%

which agrees well with the typical analysis of the Lockington core samples, given in Table 2. The low moisture content shows that the core gives up its surplus water within a comparatively short time; there is no doubt of that after an examination of the exposed core in place. Similar samples taken at Englewood had a moisture content of about 24 per cent. The weight per cubic foot was about 121 lb. A large number of pieces of core material at Englewood, some of which were 1 cu. yd. or more in size, were found at the foot of the slope after the wash-out, which showed the toughness of the core material. The Englewood wash-out occurred in a section of the dam that had been built up as much as 30 ft. vertically in 1 month, so it is evident that consolidation takes place rapidly, even when construction progress is fast. In the face of the evidence offered by these core exposures, there is no doubt that consolidation of these cores has progressed at a highly satisfactory rate.

How the Core Gives Up Its Surplus Water

There is difference of opinion as to how the surplus water escapes from the core. Experience at the Conservancy dams indicates that this process is more rapid than could occur by ordinary filtration action. It is very likely that some of the water drains out laterally, but there are indications that the

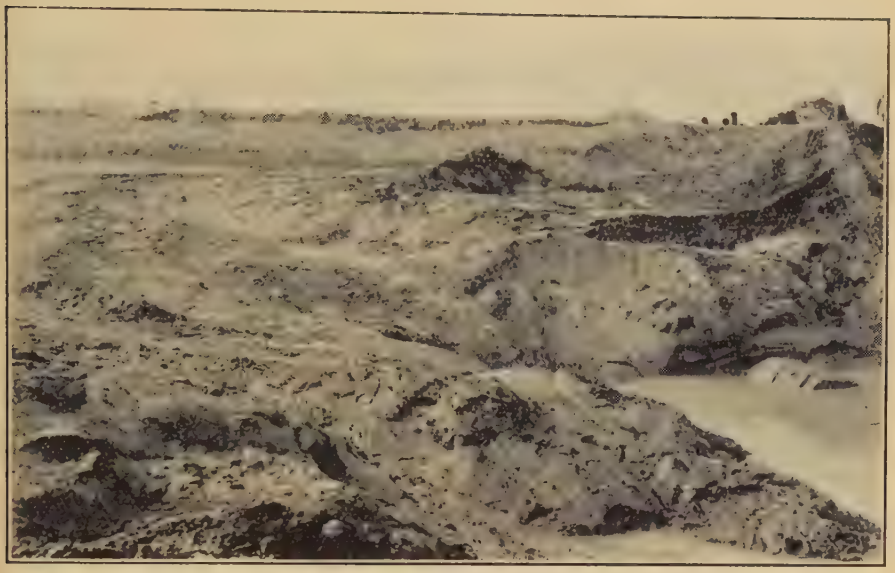

FIG. 7.—CORE OF HUFFMAN DAM, MIAMI CONSERVANCY DISTRICT.

FIG. 8.—CORE OF LOCKINGTON DAM, MIAMI CONSERVANCY DISTRICT.

FIG. 9.—CORE OF ENGLEWOOD DAM, MIAMI CONSERVANCY DISTRICT.

FIG. 10.—TOP OF LOCKINGTON DAM AFTER WASHOUT DUE TO OVERFLOW OF CORE POOL.

FIG. 11.—CORE SPRINGS OF LOCKINGTON DAM, SHOWING HOW SURPLUS WATER IS FORCED UP THROUGH CORE.

greater quantity is displaced and forced up, vertically, by settlement of the solids and by pressure due to the superimposed material. In many places, where pumping had been discontinued temporarily, innumerable little springs might be seen in the bottom of the core pool, where water was being forced up by piping. Fig. 11 is an illustration of this at one of the dams, where the core pool had been drained preparatory to topping the dam.

There is further evidence that comparatively little of the water in the core material escapes laterally. While the dams were being built, seepage water was always in evidence at the toe of the porous slopes. This flow was greatly diminished at times when the dredge pumps were shut down and the level of the core pool had dropped 1 or 2 ft. During the construction of hydraulic-fill dams in the Arid West, vegetation will grow at the foot of the slopes while hydraulic fill is in progress, but as soon as the work is discontinued, the vegetation will die for lack of water, the explanation being that along the edge of the core pool, immediately below the surface, is a strip of porous slope material not yet silted up by the core; as soon as the water surface drops below this unsilted material, little water escapes through the slopes, and the core pool holds the remainder of the water for a long time. The little springs are still in action, however, and the consolidation of the core goes on, forcing the surplus water to the surface, as illustrated in Fig. 11. It is difficult to explain the rapid consolidation of the Conservancy cores in any other way.

Gravel and Sand Sloughing into Core

Once at Huffman and several times at Englewood, trouble was experienced by the gravel and sand sloughing or raveling into the core, its presence being detected by the rod soundings. In every case, this happened when the core material was not brought up fast enough to correspond with the building up of the outside slope material, and it could always be prevented by keeping the core well filled with mud. The coarser material takes a fairly steep slope next the core pool, and each layer overhangs the one below. Water pressure is not sufficient to hold it in place. If the core pool is kept well filled with mud, however, it consolidates fast enough to keep the coarse material from sloughing in, and here, again, the advantage is apparent of having a surplus of fines in the core material so that some of the finest may be wasted.

A concentrated flow down the beach from the dredge pipe will tend to carry a tongue of gravel and sand out into the core. This may be prevented by shear boards to break up the concentration of flow, or by a floating timber held in by ropes close to the shore of the core pool, at the point where the flow enters the pool. By building up the dam in lifts of not more than 2 or 3 ft. at a time, this trouble may be avoided more easily.

Progress of Construction

It may be of interest to give a general idea of the rate of progress on the Conservancy dams. At Germantown, with one drag-line machine in the borrow-pit, and one dredge pump, an average month's work was about 60 000 to 70 000 cu. yd., and during one month, the output reached 91 500 cu. yd. At Huffman,

with similar equipment, the rate of progress was about the same, although at Lockington, the progress was somewhat less. At Taylorsville, with four giants in the borrow-pits and two dredge pumps, the highest monthly estimate was 107 000 cu. yd. At Englewood, with three drag-line machines in the borrow-pit, and two dredge pumps, the quantity of material placed in the dam often reached 150 000 cu. yd. per month, and during one month it was 180 000 cu. yd.

DISCUSSION

GEORGE L. DILLMAN,* ESQ. (by letter).—Tightness and safety are two entirely separate problems in dams, in this case, earth dams. Tightness can be, and often is, obtained at the expense of safety. This is the case when the most nearly water-tight surface is too far down stream. Safety can exist in leaky dams.

The attempt to solve the two problems by one idea has resulted in cores of sluiced material, puddled clay, masonry, concrete, and steel. All central cores are a mistake, not always failures, but always uneconomical.

If sluicing is a cheap method of moving material, it is advisable. It was used largely in the construction of the San Leandro Dam, for many years the highest earth dam in the world. Sluicing was well known to Chabot, and had been developed by hydraulic miners in California. Dikes were not made to confine the sluiced material, but to train it. The fineness or coarseness was not measured, nor thought of. The sluiced material was mixed with scraped and dumped material, filling most of the interstices therein.

A great deal of the Gatun Dam was sluiced, the original intention being to haul the material of the Culebra Cut by trains, but it was found to be cheaper to waste the Cut and borrow for the dam. The core at the site was cheap, irregular, and enormous, and its economic width was not discussed, probably not thought of.

Slides have occurred during the construction of many of these soft cored dams (sluiced material or puddled clay). Such troubles come from weak dikes or overtopping. The cores are viscous, and the dikes must act as dams to hold them. Two dams are built in order that one may result. If construction can continue to completion, safety is certain as far as cores are concerned. Contact and wasteways are other questions.

The fineness or coarseness of material is inconsequent, provided there are enough fines to enclose the coarse. Cyclopean concrete is not more pervious than other masonry, when it is properly placed; this is also true of core material. Water drains faster from coarse material; therefore, this question only concerns the rate of progress. Drainage may be facilitated by tiling.

Although an insignificant percentage of coarse sand and gravel went into these cores, it does not follow that 10, 15, or even 20% of gravel would be detrimental; because 78% of fine silt and clay was the average content, it does not follow that 100% would have been bad. There seems to have been no reason for wasting some material and borrowing other, except to obtain a pre-determined mixture, for which the reason is not apparent or real.

The reason for thickness is only to insure continuity. A foot in thickness is as valuable for tightness as that used. Tightness is not essential to safety, but drainage is essential. When the most nearly water-tight surface is at or near the up-stream face, any reasonable section will make a safe dam. On streams like the Miami, it would be impracticable to keep them leaking for many seasons. The Mackay Dam on Big Lost River in Idaho is cited, the foundation of which and the adjacent material for construction was lime-

* Engr., San Francisco, Calif.

stone gravel. Any scheme for making the dam tight would have made the cost prohibitive. There was no clay, little silt, hardly any sand and cement was out of the question, therefore, the verdict was "let her leak"; and it did leak, through, under, and around the dam. The leakage was 160 cu. ft. per sec. the first time the reservoir was filled, but the dam was always safe; the leakage was no more than had to be passed for prior rights, so no loss occurred.

The only care in construction was to have the up-stream face the most nearly water-tight surface. The bulk of the material was excavated by steam shovel and moved to place in cars. A small part was segregated, the fines being placed on the up-stream face and the coarse on the front or down-stream face. Some rock was quarried to increase the rip-rap, and a bar of material somewhat finer than the average was borrowed to increase the fines of the up-stream face.

Of the thousands of springs developed, not one shows roil; there was never any piping; there was no danger. Continued measurements show that the dam is tightening. The reservoir was first filled in 1918, and has been filled annually since.

The Miami dams are not criticized as being unsafe, but as being uneconomical. Absolute tightness was unnecessary, and the attempt to get it cost much money. If there is leakage past the cores, they will never become tight. The up-stream face will gradually become the most nearly water-tight surface, and safety will be increased thereby. An inexpensive segregation of sand and gravel (the material of the body of the dam) is all that was necessary to insure safety. After that, tightness could not have been prevented.

Nothing is said by the author about performance, although two of the dams were completed before the paper was written. Another point of importance, wasteways, is not discussed. It is to be hoped the author will allude to them in his closing discussion.

Allen Hazen,* M. Am. Soc. C. E. (by letter).—This paper is a statement of what can be done with material of hard-grained particles from the terminal moraine of the glacial period of Ohio. It will be well to remember that other physical properties may be found in other stock and in other parts of the United States.

The author speaks of the decided advantage in having a surplus of core material in the stock so that a part, at least, of the finest of that material may be wasted. This means practically that the effective size of that which remains and forms the core will be increased. Mechanical analyses of representative samples of core material are given, but the effective size has not been computed, and there is difficulty in doing so, because the finest size of separation is greater than the effective size, and results can only be reached by extrapolation. The writer has plotted these results on logarithmic probability paper and, by extending the line downward, he finds that the effective size of the average of the samples is 0.0025 mm. The author does not state whether the mean or the

* Cons. Engr. (Hazen, Whipple and Fuller), New York City.

greatest diameter was used, in describing the size of the particles. Assuming that the mean is intended, this material is coarser than the material from cores of some other dams that the writer has examined, and shows the beneficial effect of wasting some of the finest particles. In addition to the greater grain size, there are differences in physical properties between the hard-grained particles of glacial drift and the much softer particles resulting from the disintegration of soft rock in other places.

The voids in the core material are not stated by the author, but he does give data from which their amount may be inferred.* Proceeding in this manner, it appears that the Lockington core had from 42 to 44% and the Englewood core 46% of voids. The weight per cubic foot of wet core material stated to be 121 lb., indicates 43% of voids. These results are consistent and indicate a slightly greater degree of consolidation than was found at the Calaveras Dam in California at the time of the slip.

The use of, and the results obtained by, the Goldbeck pressure cells are interesting. The writer doubts whether useful deductions can be safely made from these results at this time. In the first place, an idea of the degree of accuracy of the gauges may be obtained from the charts for vertical pressure. The vertical pressure certainly exists to the extent of the full weight of the material, and if the gauges were correct the indications would correspond to that weight. The figures taken from the diagrams do not appear to do this.

The observations at Germantown on March 20th, 1920, showed a pressure of 20 lb. at a depth of 26 ft., indicating 92% of the computed weight of the material at 121 lb. per cu. ft. At a depth of 53 ft., a pressure of 30 lb. was indicated, or only 68% of the computed weight. At Taylorsville, on November 26th, 1921, a gauge at a depth of 67 ft. showed 50 lb., or 89% of the computed weight of the fill above it. The indications are that all the gauges under-register and that variations occur between the several gauges, that are not accounted for. For horizontal pressures, the gauges indicated somewhat less than vertical pressures at the same points for the greatest depths.

When core material is soft, the horizontal and vertical pressures are equal. As the work of construction proceeds, and more material is placed above, and as the core becomes consolidated, the horizontal and vertical pressures presumably remain more or less equal, unless something happens to disturb the equilibrium. A movement or flow of core material would be caused either by an increase or decrease in horizontal pressure. To cause movement, the change in pressure would have to be sufficient to overcome whatever degree of stability had been reached to that time. Between these limits, far apart, is a broad range of pressures through which the material is always stable. Tests that showed the actual horizontal pressure at the moment when movement or flow began, would certainly be interesting. The pressure that exists with no movement would seem to have no significance; it might be anything within the limits of stability, and probably would not differ widely from the vertical pressure.

The description of experience with actual breaks in the sides of dams, during which the core material was eroded and exposed, is one of the most

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), Table 5, p. 1792.

valuable parts of the paper. It shows that the core material had attained a considerable degree of stability at the times of the breaks, but it must always be remembered that stability of such material is dependent, in considerable measure, on the height of the exposed face. Soft material is stable as long as the height is not great. Every material has its limit of height, and the greater the cohesion the higher the limit. At the Calaveras Dam, it was found that a material that would stand at one height, would flow almost as a liquid at a greater height.

The hydraulic method of dam construction is a most valuable one and the author has made an important contribution to the useful knowledge of it.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—There are few words subject to more interpretations than "soil", and "silt". Weight as well as fineness are distinguishing qualities. An analysis for size and a determination of specific gravity of the material when dry and when saturated would convey quite a definite idea of the core material and could be used for comparison in other parts of the world. Is a combined size and weight table available? The author has "broken ground" in core-wall investigations and should have credit for incurring the expense necessary to determine the changes that a hydraulic fill undergoes. The writer would make a suggestion in regard to a pressure cell.

On the center line of the core, erect a cylinder or hollow mast, 28 or 30 in. in diameter, of thin metal, say, $\frac{3}{16}$ in. in thickness, or thick enough to withstand the exterior pressure, made up of sections suitable for riveting and caulking as the core height increased. Each section should be water-tight when in place and have an iron ladder on the inside, with one or more narrow shelves, suitable for a foothold. Depending on the length of the section, there should be one or two openings, 5 or 6 in. in diameter, over which a short cell holder or frame of metal should be riveted and made water-tight. The holder should project only 2 or 3 in. The cell itself would have two walls of different material, the outer one of durable rubber or canvas, reinforced by an added thickness at the center and held to the cell frame by a water-tight connection, and the opposite or inner wall would be of strong, clear glass fitted securely in the holder and carrying a short threaded tube. An observer would have good footing in the mast and with a pressure gauge and a small air-pump could force the outer cell wall against whatever pressure there might be, watching the movement through the glass disk. The pressure could be noted and also the return motion of the opposite wall or diaphragm as the air pressure was permitted to decrease. This operation could be repeated several times to obviate instrumental error, and the results from another cell below or above could be used as a check.

The vertical pressure could be ascertained by making a very short bend in a cell frame, and having the opening for the cell holder elliptical, to permit of direct observation. When the dam was completed and the top of the mast sealed, an opportunity would still exist to examine the cells and record the resistance to pressure. A number of years might pass before the cell wall would fail, especially if one or two were to be made like a piston of more permanent

* Civ. and Hydr. Engr., New York City.

material; and after that samples of the core could be secured. One mast might be regarded as satisfactory, if material of inferior quality was to be deposited around it, that is, it would be natural to conclude that if the poorer part of the fill was shown to be safe against movement, the better part would be safe. The estimated cost of an apparatus of this kind might be compared with the cost as conducted at one or more of the dams named in the paper.

In massive construction in which there was danger of lateral motion of core material, parts of a mast could be used from a raft carrying a suitable framework and equipment for thrusting down and lifting up a section of reasonable length. This could be used at more than one place in the pool. It might result in the abandonment of cell construction and the use of a plain plunger of appropriate size and weight that could be rapidly settled or lifted while the stresses in the control apparatus were recorded and the character of the material thereby estimated.

THOMAS H. WIGGIN,* M. AM. SOC. C. E. (by letter).—This paper records the results of observations and experiments on five ably constructed hydraulic-fill dams. The use of Goldbeck cells for observing the vertical and horizontal pressures in the cores makes the record unique and it is to be hoped will mark the beginning of a new era in the study of hydraulic-fill dams, which have been under a heavy cloud since the accidents at the Calaveras and Necaxa Dams.

The writer visited the Gatun Dam in 1910, during its construction, and recalls the doubts then expressed by the Division Engineer, based on his tests, as to whether the core would ever solidify. The exceedingly flat slopes of that dam made the question of the consolidation of the core relatively unimportant. However, it would be valuable to have the Gatun data added to this discussion.

In 1919, the writer inspected the San Pablo Dam, then partly constructed, and made some computations on various assumptions for John R. Freeman, President, Am. Soc. C. E., who had been consulted with respect to the dam. A more indeterminate problem than these of hydraulic-fill dams cannot well be imagined. The uncertain factors are:

(*a*) Pressure of the core, which may be anywhere from 0.8 to 1.6 times the pressure of water of equal depth.
(*b*) Coefficient of friction of the retaining embankments which may be anything from 0.2 to 0.8, probably in this case nearer the lower limit than the higher, particularly for certain decomposing argillaceous rock where located below the line of saturation.
(*c*) Position of line of saturation on down-stream slope, reducing the weight of the embankment to which the coefficient of friction would be applied, as a factor, in determining resistance to core pressures.
(*d*) Level of water up stream, which affects not only pressures from the core, but also weight of material, due to submergence.
(*e*) Proper factor of safety which might be assumed as anything from 1.5 to 3.

An indefinite number of possible designs can be computed with slopes varying from about 1 on 1 for the most favorable assumptions, a factor of

* Cons. Engr., New York City.

safety of 1.5 to about 1 on 15 for least favorable assumptions, and a factor of safety of 3. Added material beyond these slopes is necessary where the core at the base is wider than the embankment at the top of the core, as, in this case, a part of the embankment is floated on the core.

Probably in no other branch of hydraulic construction is the element of judgment greater. The need for economy in first cost contends strongly with the moral and economic necessity for safety, and the engineer's position is not a comfortable one, particularly when the physical properties of the available materials have special elements of doubt.

Only an accumulation of such observations and records as those contained in the author's paper will reduce these uncertainties and furnish reliable constants for safe design, not to mention constants for the more refined determinations as to how far it will pay to waste fines, in order to reduce core pressures and hence be able to reduce the size of embankments.

Experimental data on cores will be of limited value in design unless reliable methods of classifications of core material can be devised. Mechanical analysis giving the size of the grain is the only method used thus far and probably gives the most important single criterion. Graphs are more easily followed than tables of analyses and a combination graph showing complete analyses of all core material tested by the Miami District and other observers would be of great value. The need of such data is referred to subsequently, in connection with certain cores.

Experimental data are also needed on the friction of embankment material. It is just as necessary to know the available resistance as to know the disrupting pressures. The San Pablo experiments* made by G. W. Hawley, M. Am. Soc. C. E., at the suggestion of Allen Hazen, M. Am. Soc. C. E., are the only ones known to the writer. They were on 14-in. cylindrical sections. At least, some experiments on much larger sections, with comparative tests on small sections of the same material, are needed before placing too much faith in the small-scale experiments. The effect of moisture on coefficients of friction is another phase requiring tests, and the effect of partial suspension of material by water below the line of saturation adds to the complexity of the problem.

The author gives stimulus to the discussion begun with an earlier paper,† as to the degree of coarseness of core material required in order that consolidation may proceed promptly enough to permit lightened design of embankments. Mr. Hazen suggests an effective size of 0.01 mm. (that is, 10% to be finer than 0.01 mm.) as fine enough to insure water-tightness, coarse enough to drain, and demonstrated at San Pablo to be practically attainable (although, in the writer's opinion, with much wastage of fines). In his paper, Mr. Hazen contends that the ordinary cores, which he found to have generally an effective size of about 0.002 mm., do not consolidate much in the comparatively short time during construction.

Mr. Paul believes the Miami cores to have consolidated satisfactorily, although much finer than 0.01 mm. He concludes that "a fairly high per-

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 1728 and 1818.

† "Hydraulic-Fill Dams", by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Vol. LXXXIII, (1919–20), p. 1713.

centage of extremely fine material in the core is not objectionable, provided the material is properly graded."

He does not use the water-filtration term, "effective size", but the analyses of the Miami cores in Table 2 show 22% on the average finer than 0.005 mm., and this is probably not far from having 10% finer than 0.002 mm. In other words, the Miami cores seem to be of about the same fineness at the 10% point as the cores of the much discussed Western dams. The author mentions analyzing cores of these other dams, and in this connection the graphs showing the complete analyses that are available, are required, as explained previously. In fact, such a compilation is needed to understand the author's conclusion, that proper gradation is an offset for a considerable percentage of fines.

The writer has been interested in pursuing further, as well as may be without complete analyses, the evidence as to what real difference existed between the material in the Miami cores and that in cores of the Western dams.

As to penetration tests, a 6-in., cast-iron ball sank in the Calaveras core from 50 to 60 ft. at the maximum and in the Miami cores only 16 ft. Similarly, a 1½-in. pipe, pushed down by two men, penetrated 80 or 90 ft. at Calaveras and only 30 ft. at Miami.* However, Mr. Hazen, in commenting on these records, stated his understanding that, at the time of the great slip, penetration in the Calaveras Dam was no greater than in the Miami dams. This evidence is not very conclusive as to showing differences in the core material.

As to the appearance of the exposed cores, those at Calaveras and Necaxa were firm after they ceased to move. Those at the Miami dams were firm when exposed at depths of 30 ft. There is something peculiar, as was stated in the discussion on Mr. Hazen's paper, previously mentioned, in the apparent dynamic fluidity which comparatively firm substances have when once cohesion or frictional stability is overcome and they begin to move. It may be that the Miami cores, if started at greater depth, might have shown mobility in the superimposed parts.

As to comparative pressures exerted by the cores of the Calaveras and Necaxa Dams and the Miami cores, unfortunately, one is reduced entirely to speculation in the case of the former two, not having Goldbeck cell records as in the latter case. Mr. Hazen notes† a coefficient of friction of 0.2 for the Calaveras embankment, which was pushed out. This assumes the core to be a fluid of its actual weight. The writer's figure using similar assumptions was 0.3, evidently showing a different section assumed from the records of the slip. D. C. Henny, M. Am. Soc. C. E., computed a maximum coefficient of 0.7 for the same dam.‡ The writer's figures for the Necaxa Dam were about 0.2 for the theoretical section and 0.6 for the actual worst section, as described by one of the constructing engineers.

From the Goldbeck cell indications, the Miami cores had horizontal pressures on the average such as would be due to a fluid weighing about 50 lb. (Fig. 6) to 70 lb. (Fig. 4) per cu. ft. If the Necaxa core at failure

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 1797 *et seq.*

† *Loc. cit.*, p. 1728.

‡ *Loc. cit.*, pp. 1760–1761.

had exerted no greater pressure than these Miami pressures, the coefficient of friction at the worst section would have been 0.3 to 0.4 and the Calaveras coefficient correspondingly 0.15 to 0.20 (or 0.35 to 0.50 by Mr. Henny's results). It seems doubtful whether failure would have occurred at these figures, which argues that the pressures in the Necaxa and Calaveras cores were probably higher than the pressures indicated in the Miami cores by the Goldbeck cells.

On the other hand, even if the Miami cores had exerted a pressure equal to a fluid weighing 100 lb. per cu. ft., the embankments would not have been breached unless their coefficients of friction had been less than about one-third, which seems improbable for well-drained gravel. The well-nigh perfect control of core widths eliminated at the Miami dams the greatest element in the failure of other dams.

Considering the Goldbeck cell indications, it may be possible to learn something about them from the internal evidence of the records themselves and by studying limits of possible errors. Mr. Hazen has suggested that the reaction from the small isolated area of the cell might be carried by a pyramid of earth, as is the case in small area tests of the bearing value of soils. This is undeniably possible but, as has been stated by the author, would cause the Goldbeck cell indications to be too high—an error on the safe side in the study of core consolidation, and an error tending to a minimum the more fluid the core. The small deflection, namely, 0.0001 in., required to break the electrical contact, favors accuracy; studies of soil elasticity would bear on the question.

The most direct internal evidence as to the Goldbeck cell indications should be that relating to vertical pressures. These pressures are presumably determined by the weight rather than the degree of solidification of the superimposed material, and the weight is fairly well known. The writer has converted the vertical pressures given in Figs. 4 and 6 into average weights per cubic foot of superposed wet core material, as shown in Table 3.

TABLE 3.

Date.	Depth, in feet.	Vertical pressure shown in diagrams, in pounds per square inch.	Equivalent weight of core per cubic foot, in pounds.
	From Fig. 4, Germantown Dam.		
September 10, 1919	12	6	72
September 25, "	18	11	88
October 23, "	25	18	104
November 12, "	29	21	104
December 5, "	33	22	96
January 27, 1920	41	23	80
March 3, "	41	22	77
April 28, "	48	27.5	83
May 20, "	26	20	111
May 20, "	54	30	80
	From Fig. 6, Taylorsville Dam.		
July 6, 1921	24	10	60
July 28, "	33	23	100
October 27, "	59	45	110
November 26, "	70	52	107

The results in Fig. 4 at the Germantown Dam where the trestle installation was used, seem to be erratic, often giving a lesser weight per cubic foot at greater depths, although the average weight should be greater as the depth increases and the water is squeezed out of the lower layers—unless drainage should be so complete as to leave voids unfilled with water, which is unthinkable. It is more plausible to assume the material to be supported in part at times by the wooden trestle. This trestle, if constructed of timbers even as large as 4 by 4 in. (not marked on Fig. 4), could take the load in question.

The Taylorsville (Fig. 6) results for vertical pressures are more consistent, 107 lb. per cu. ft. is the weight of saturated earth having about 56% of voids. If the lower material is conceived to have 50% of voids and to weigh 114 lb. per cu. ft. and the top layers to weigh 100 lb. per cu. ft., the average weight would be about 107 lb.

Similar computations of the horizontal pressures show that, whereas those at Taylorsville (Fig. 6) never reached, at any depth, the intensity of hydrostatic pressure, those at Germantown (Fig. 4), above the depth where the curves turn sharply downward, corresponded with the pressure of a fluid weighing from 84 to 104 lb. per cu. ft., averaging about 96 lb. At greatest depths, the pressures at Germantown were, curiously, on nearly every date just a little greater than hydrostatic. In general terms, the horizontal pressures at the Germantown Dam were much higher in absolute value, and more so when considered as functious of the vertical pressures, than those at the Taylorsville Dam. They also showed a sharper reduction at lower depths, but even there did not go as low as those at Taylorsville.

These differences could be explained by assuming (*a*) the existence of a much more fluid core at Germantown (at the trestle) than at Taylorsville (near the wall); (*b*) that the pressures at greater depths at Germantown are reduced by the trestle; and (*c*) that the trestle had more effect on the horizontal than on the vertical pressures.

The writer confesses to some perplexity resulting from all the considerations previously given, as to whether the Germantown core differed so greatly after all from the Calaveras and Necaxa cores. The question also suggests itself whether the Goldbeck cells in the Taylorsville Dam were placed so as to give representative pressures in the mass of the core; that is, whether this core also might have given high horizontal pressures away from the concrete wall and wasteway. The writer is not disposed to doubt the substantial accord of the Goldbeck cell indications with the actual pressures existing in the core at the place where the cell is located.

The complete mechanical analyses of the Calaveras and Necaxa cores, if available, plotted to one scale with those of the Miami cores, would throw light on these questions, but the writer fears that there may still be more obscure and less easily recognized differences, such as those suggested by studies of colloidal suspension (see, for example, the summary on this subject in the Progress Report of Special Committee on Bearing Value of Soils).* In the future, some one may add a little of a chemical—an electrolyte—to the

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 547.

sluicing water and precipitate core material which otherwise would be unsuitable. Even now, fear of some disturbance of the colloidal condition might well cause distrust of small-scale experiments on core material.

There is left the question of lateral drainage. A small percentage of reduction in the core moisture near the critical point is probably sufficient to increase greatly the stability. Might not such a reduction take place without noticeable wetness at the toe of the slope? Is it more difficult to conceive of such moisture going laterally than vertically, in either case, through thick layers of fine material? Must it not, in any case, go laterally to connect with the springs? The pressure of superimposed material steepens the hydraulic gradient as well for lateral as for vertical escape of water. Impervious, clayey embankments would retard or prevent lateral escape. The San Pablo tests, showing no flow through perforated casing pipes into test borings, are hard to harmonize with the idea of lateral drainage. It is true that the velocity of drainage water would have to be increased greatly to concentrate on the boring and through the still smaller slotted areas. The difficulty of getting water to well strainers in fine sand is one often experienced. With core material, which is several degrees finer than fine sand, practically complete plugging might be conceived as against flow, but it is hard to explain a well standing empty for weeks.

All the arguments for the efficacy of lateral drainage tend to contradict the previous suggestions that high horizontal pressures may have persisted at the Germantown Dam and perhaps at the Taylorsville Dam away from the wall; for conditions favoring horizontal drainage were ideal in the Miami dams. There is still much to learn about the drainage question.

The writer will await with much interest the author's experienced judgment and data on the various academic questions raised. The author and his colleagues on the Miami Conservancy work are to be congratulated on their skillful design and construction and thanked for taking the first steps and pointing the way to the accumulation of real data on core pressures.

H. S. R. McCurdy,* M. Am. Soc. C. E. (by letter).—The writer's association with the construction of the Englewood Dam has so impressed on him the importance of a correct handling of the core feature of a hydraulic-fill dam that he cannot refrain from emphasizing some of the important factors.

In any hydraulic-fill dam constructed of glacial materials three classes of gradation are prominent, namely, the porous outer part, composed of a mixture of sand and gravel, the central core of finely divided particles, highly impervious, and the "twilight" zone composed of sand, between the first two.

The outer part requires little attention beyond the assurance that it is properly porous and sufficiently heavy to resist the lateral thrust of the core. Properly porous, in this case, means simply that the hydraulic gradient of the water draining from the pool and the core is sufficiently steep to lie below the slope of the dam; otherwise, sloughing is likely to occur. At four of the Conservancy Dams this outer material was a nearly ideal mixture for concrete aggregates. Indeed, at Englewood, excellent concrete was made for the

* Div. Engr., The Miami Conservancy Dist., Englewood, Ohio.

pump-houses by the use of bank-run material, shoveled from the slopes of the dam. There was little variation of gradation from the outer slope to the core margin.

The "twilight" zone, composed of sand of various sizes, needs no particular attention. The core material, however, requires the most painstaking and constant investigation and control. Two things which need particularly to be guarded against are likely to happen: First, and more serious, is the encroachment of the material of the porous outer part of the dam into the zone that should be occupied only by impervious core. This is caused by a failure of the core to consolidate with sufficient rapidity to withstand the thrust of the outer gravel tending to invade it. Second, and of somewhat similar effect, is the washing of sand and gravel into the core by the action of the sluicing water, as it enters the pool. Although these matters require constant vigilance, it is not the writer's intention to create the impression that they present any great difficulties nor that, with intelligent supervision, there need be any fear as to the successful outcome.

To insure that the core shall consolidate with the desired rapidity, two conditions are necessary: First, the material in the borrow-pit must contain a sufficient proportion of "fines" to enable the more finely divided particles to be wasted; and, second, the engineer in charge should realize that, on occasions, he must be prepared to waste good dirt, when, in his judgment, the core is not properly consolidating. No attempt at economy is more short-sighted than to try to conserve those particles in the core that are too fine for the work.

It is one thing to stipulate that the too fine particles shall be wasted and quite another to determine in practice just what constitutes this critical size. So many factors enter into the question that no general law can be laid down. Fundamentally, the size of the particles is the criterion, but here, again, one is confronted by the fact that the same size would not apply to all dams. The speed with which a dam is built vertically has a great bearing on the subject. Obviously, for a dam that is being built rapidly, less time for consolidation would be allowed and coarser particles in the core would be required than for one being built more slowly. Laboratory analyses might give information on the subject, but the difficulty of reproducing working conditions renders this method of little more than passing interest. As far as the writer knows, no better method of testing the behavior of the core from day to day has been devised than by taking soundings with a rod or pipe. This method is simple and is positive if it is used by an intelligent observer. No difficulty is experienced in identifying layers or deposits of sand or gravel, if any are present within the core, and the rate of consolidation, to the point where the rod meets refusal, can be accurately determined.

Although it is essential that the core be kept well up to the surface of the pool, it must not be thought that because this is so, all necessary conditions are fulfilled. Many times at Englewood the core had the appearance of being full when it was known that the material seeming to constitute an excellent core was too fine to consolidate properly. The solution is to use a larger proportion of fine material and to waste more core until the stage is

reached at which proper consolidation is taking place. This point cannot be emphasized too strongly. Proof more conclusive than mere appearances must be had that proper consolidation is taking place, otherwise there is every likelihood that the core zone will be invaded by the outer porous material. This might take place to a certain degree and not be serious, provided a sufficient thickness of impervious core is maintained at all places. Most dams are designed with a thicker core than would be needed to prevent leakage if positive assurance was to be had that the core, as constructed, conformed to the theoretical limits. This ideal condition is far from being realized in practice.

To prevent material from being carried into the core in suspension in the sluicing water, although important, is not difficult. The likelihood of this happening is an unanswerable argument against too narrow a pool. A narrow core may be sufficiently impervious to prevent seepage of impounded water through the dam, but it is quite another thing to construct such a core. Where small quantities of sluicing water are used it might be practicable, if desirable, to deposit a correspondingly narrow core, but where the discharge from each dredge pipe is 15 sec-ft. or more, as was the case at some of the Conservancy Dams, it would be almost impossible to prevent sand and gravel from washing across the core with too narrow a pool. This is particularly true if the beaches are formed of the coarse gravel of the Miami Valley, which often slope as much as 10% from the outer edge to the pool margin. The pool retards the velocity of the entering stream to a certain extent, but there are cases where the water issuing from the dredge pipe will cut gullies, thus concentrating the flow and causing it to enter the pool with sufficient velocity to transport the heavy material a considerable distance from the shore. A full, well-consolidated core also conduces to the deposition of gravel in the center. In itself, such a condition is ideal, but there is a tendency for the sluicing water to ride on top of the core, carrying the coarse material with it. Unless this action is too extensive, it may do no harm, as a limited quantity of gravel can be absorbed by the core without affecting its imperviousness, provided the gravel is well scattered.

Various devices are feasible for preventing this wash into the core to an injurious extent. Some wash is to be expected, notwithstanding that all reasonable precautions are taken, and to provide for this without encroaching on the core limits, the shore lines are moved out a distance determined by observation, thus widening the pool. The preventive lies in the use of baffle-boards placed across the path of the sluicing water in its course down the beach to the pool. The efficacy in the use of baffle-boards lies in their ability to scatter the flow over greater areas. At Englewood, it was found that baffles on the beach were not entirely satisfactory. Although they would distribute the flow temporarily, there would be a tendency for it to concentrate again below the baffles and recover all its erosive characteristics. Perhaps this was due largely to the volume of water and to the steepness of the beach. By placing baffles in the water at the edge of the pool, however, the difficulty was solved. These baffle-floats were constructed of 6 by 8-in. timbers, 16 ft. long, placed parallel and 2 ft. apart and tied across with 2-in. plank. A line

of 2-in. sheeting, extending 2 ft. 6 in. down into the water, was spiked to the outer timber. The floats were anchored near the shore by ropes tied to bars thrust into the beach. Any width of delta could be covered by attaching the floats end to end.

An excellent opportunity to study the core at Englewood was unintentionally afforded when, during the night of October 25th, 1921, the core pool overflowed the levees and gullied out the side of the dam, exposing a face of about 30 ft. of core. The material had been in place only a few weeks, yet a state of consolidation had been reached that permitted about 750 000 gal. of water to cascade over the face of the core and remove less than 1 000 cu. yd. Not more than 10% of the material eroded was from the core, and chunks of this, 1 cu. yd. or more in volume, were tumbled down the side of the dam for several hundred feet and yet remained intact. After the water had ceased flowing, no movement of the core was noticeable, although an almost vertical face of 30 ft. was exposed.

If it was possible to ascertain, with any degree of certainty, the frictional resistance of the outer parts of an hydraulic-fill dam and the lateral pressure of the core material, the design of the cross-section would approach an exact science. These conditions at present can only be estimated, allowing a sufficient factor of ignorance to be conservative. However, the use of the Goldbeck pressure cells, as adapted by Mr. Paul to conditions in the Conservancy Dams, marks a distinct step forward in the investigation of core pressures. To the writer's knowledge this is the first attempt at anything of this nature. Although results in some cases were not all that could be desired, many valuable data were obtained as to the rate of consolidation. One thing clearly brought out was that the proportion of lateral to vertical pressure decreased positively and progressively and that the time element, coupled with the expulsion of water due to the surcharge, effectively robs the core material of its hydrostatic characteristics.

Joel D. Justin,* M. Am. Soc. C. E. (by letter).—The term, "hydraulic-fill dam", has often been used to include all dams, in the construction of which water has been used, at any stage of the process, for loosening, transporting, and distributing the materials. The writer believes this definition is too broad, as it includes two different types of dams, and he, therefore, suggests the following definitions, in order to differentiate between them.

Hydraulic-Fill Dam.—An earth dam in the construction of which the materials are transported and distributed by water to their final position in the dam.

Semi-Hydraulic Fill Dam.—An earth dam in the construction of which the materials are transported on the dam by other means than water, dumped within the section of the dam, and some of this material moved to its final position in the dam by the action of water.

The Calaveras, Necaxa, Linville, Standley Lake, and the Gatun Dams were of the semi-hydraulic fill type. In the construction of this type, trestles are generally placed near the toes of the dam, and the material

* Chf. Engr., Depts. of Hydro-Elec. Power Plants, Water Supply, and Sewerage, The Ludlow Engrs., Inc., Winston-Salem, N. C.

dumped from these trestles is sluiced into a central pool by the action of giants. Some of the material dumped from the trestles is not moved by the action of water. The slides of large masses of the material composing the dam have been confined almost entirely to the semi-hydraulic fill type of dam.

The outside portions of dams of this type, frequently consisting of car-dump fills, are often less pervious than the sluiced material immediately inside. They are not self-draining and cannot aid in the draining of the core. The method of construction of the hydraulic-fill dam, however, insures a relatively pervious and free-draining outer section, for as soon as the water is released from the flumes at the toes, it will begin to drop the coarser materials.

The Miami Conservancy dams are true hydraulic-fill dams; although much of the material was excavated from borrow-pits and hauled to the site in cars, it was broken up by giants and pumped through flume pipes from which it was released near the toes to flow into the central pool.

The importance of self-draining, relatively pervious, outer sections was appreciated by the engineers of the Conservancy District, and they were willing to go to some expense to obtain them, as it would have been cheaper apparently to have used the semi-hydraulic fill method and had car-dump fills at the toes. Even if most of the drainage from the cores takes place by vertical crater action, such action is not confined to the part of the core in view, but is also doubtless taking place in the parts of the core covered with the pervious outer sections of the dam. Therefore, whether the main drainage of the core is upward, downward or sidewise, the importance of pervious outer sections is just as great.

The author states that in the Conservancy dams the width of the core pool was equivalent to the remaining height of the dam to be built. This rule has evidently worked well with these dams, and there seems to be no sound reason why it should not be considered standard practice where the method of construction is the same and the materials are similar; that is, where the core material does not contain more than 35% of clay (0.005 mm. or less). It should be remembered, however, that Allen Hazen, M. Am. Soc. C. E., in his study of hydraulic-fill dams,* found that when the central pool was kept so small that the material deposited did not have an effective size of less than 0.01 mm., excellent stability of the core was quickly attained. Mr. Hazen's criterion is more severe than that of the Conservancy dams, because from 20 to 25% of the core material of these dams would have had to be wasted in order to comply with it.

The pressure-cell experiments described by Mr. Paul constitute an important contribution to engineering knowledge, and it is hoped that the observations will be continued until it is evident that a final condition of equilibrium has been reached. If careful determinations of the average weight per cubic foot of the core material had been made when the pressure-gauge readings were taken at the various depths shown in Fig. 6, a useful check on the accuracy of the vertical pressure readings could have been

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1713.

obtained. The weight per cubic foot of core material in the Englewood Dam is stated as about 121 lb. per cu. ft. in one case.

If the material in the core of the Taylorsville Dam weighed 120 lb. per cu. ft., the recorded vertical pressures in the last diagram of Fig. 6 are from 84 to 91% of the true vertical pressures. Therefore, either the weight of the material in the core is materially less than 120 lb. per cu. ft., or the cells do not record all the vertical pressure, owing, perhaps, to the bridging of pressures through the soil around the cells. This is mentioned merely to indicate the desirability of determining by further experiments the weight of the material over the vertical pressure cell, so that any necessary correction factor may be applied to the cell readings, both vertical and lateral.

Fig. 6 shows that the consolidation of the material proceeded at a rapid rate. At the end of about five months, while construction was progressing, the lateral pressure at a depth of 70 ft. was only 22 lb. per sq. in., which is only about 38% of the pressure which would be exerted by a liquid of the same unit weight (120 lb. per cu. ft. assumed). This is about the same pressure as would be exerted on a retaining wall by a filling material, the angle of repose of which is about 30° and the weight of which is 120 lb. per cu. ft. It is interesting to note that the ratio of the lateral pressure to the full hydrostatic pressure on November 26th, 1921, varied from about 0.38 at a depth of 70 ft. to about 0.47 at a depth of 23 ft., the assumed weight of material being 120 lb. per cu. ft.

These data are of particular interest to the writer as he has recently designed a 66-ft. retaining wall as an abutment for an hydraulic-fill dam. The weight of material used was 120 lb. per cu. ft. and the ratio of the lateral to the full hydrostatic pressure selected was 0.435. For this depth, Fig. 6 gives a pressure of 20 lb. per sq. in., equivalent to 0.37 of full hydrostatic pressure.

In this connection, it would be interesting to learn whether or not any special means was used of compacting the embankment just back of the retaining wall where the pressure cells were located, or whether the method was the same as that used in the body of the dam.

O. N. Floyd,* M. Am. Soc. C. E. (by letter).—The author has covered his subject so well that there is little of importance to be added. However, by way of emphasis and enlargement, the writer wishes to discuss, first the consolidation of the core due to pressure, and, second, the width of the core.

Consolidation of the Core.—The writer is convinced that any consolidation of the core that occurs soon enough to be of practical value during construction must be due to the forcible expulsion of the water from it by the weight of the material above. The principal way of escape for this water appears to be upward, through the softer overlying core material. If there is a shortage of suitably graded core material, or if the core pool is being carried too wide, a soupy sort of mixture only a little heavier than water will remain in the core. Apparently, this material is too much like mud to precipitate readily, and the mixture is too light to force the water rapidly from the lower layers.

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When pumping was first started on the valley section of the Taylorsville Dam, the up-stream toe of this section, from the proposed core line outward, had been constructed to a height of about 15 ft. by another method, so that all the pumping for the hydraulic fill in this section was done on the down-stream part until the elevation of the older embankment was reached, or for a period of about six months. This condition, together with a large cut-off trench along the center line of the dam, gave core space, in proportion to the quantity of material being pumped, more than double what it would have been under normal conditions. Frequent soundings with a 6-in. cast-iron ball and with a 2-in. round pole showed that there was quite an accumulation of this soupy material along the up-stream edge of the core pool, that weighed from 70 to 80 lb. per cu. ft., whereas the core material near the down-stream edge of the pool weighed from 115 to 120 lb. per cu. ft. Just before pumping was started on the up-stream toe, the 6-in. cast-iron ball sank into the core material from 3 to 5 ft. near the down-stream edge and about 8 ft. in the center and from 12 to 14 ft., or within 1 ft. or 2 ft. of the original ground surface, near the up-stream edge.

When pumping was started on the up-stream toe, this soft material was soon displaced by a heavier material, and as the core width was brought to normal, the cast-iron ball would sink into the core only about 2 ft. near each edge and about 3 ft. near the center. Samples of this core, taken from time to time as the water happened to be drawn off temporarily, showed a rather uniform weight of from 120 to 125 lb. per cu. ft. Samples of the sand and gravel taken a few feet back from the edge of the core usually weighed from 115 to 120 lb. per cu. ft.

There was an abundance of evidence that a large quantity of water was continuously being forced up through the core material. After a shut-down of a few days, the water in the pool was always clear enough so that the surface of the mud and the little columns of slightly muddy water could be seen rising 2 or 3 in. above the mud.

It was observed that after this core material had given up most of its water, it became impervious and prevented the escape of the water through it. Test pits along the edge of the core adjoining the sand and gravel, indicated that such a condition was soon reached in a wall a few feet thick in this part of the core, owing probably to the complete drainage afforded by the sand and gravel. Also, on exposure of any part of the surface of the core to the air and sunshine for a few days, a leathery layer a few inches thick would be formed, which, when punctured, would usually show a rather free flow of water for a time. Once while waiting (November to July), for the main line of the Baltimore and Ohio Railroad to be moved from the base of the dam, the water from the pool was drawn off, and the entire surface of the core became so firm that it was broken up with a team and plow before pumping was started again. In drying, the core settled so that along the center line it was about 1 ft. lower than at each edge. The top 3 or 4 in. became very dry and showed numerous cracks, but under this top layer there was a layer about 1 ft. in thickness, which was fairly dry and firm, but contained very few cracks. Under this, the material was a stiff mud that could be removed from a post-hole

digger only by some pounding. A number of 8-in. holes from 7 to 8 ft. deep were dug along the center line of the dam in this core and in every case, within about 3 hours from completion, the hole would overflow with clear water which would continue to flow for 18 to 30 hours. The flow from these holes was small, about such as would flow from an open notch ⅛ in. square.

The vertical progress on this section of the dam to the time of the shut-down, was only about 2 ft. per month, yet the ball and the rod each indicated that the consolidation of the core was practically identical with that of the river section at about the same elevation, which was built up 33 ft. in 1 month. The writer is convinced that within reasonable limits, time has little effect on core consolidation. There seems to be present in almost all earth a certain quantity of excessively fine light material that will not easily precipitate or give up its excess water, which must be displaced by a heavier material before any satisfactory consolidation of the core can take place. This displacement will occur automatically if sufficient fine material, properly graded, is fed into the pump or the sluice-box. When this has been done, the pressure from above will cause the consolidation of the core to keep pace with any reasonable rate of progress. At the Taylorsville Dam, during construction, the core was tested frequently with a 6-in. cast-iron ball and with a 2-in. round pole and a few tests were made by driving a 3-in. casing and cleaning out with an auger. (Samples were taken by cleaning out ahead of the casing). These tests showed that with similar borrow-pit materials the depth of the soft unconsolidated part of the core was no greater in the river section of the dam where the vertical progress was 33 ft. per month than in the valley section between the same elevations where the progress was only about 2 ft. per month.

Width of the Core.—The safety of an hydraulic-fill dam during construction may be seriously endangered by carrying the core too wide, and the cost of the work may be increased unnecessarily if it is kept too narrow. As stated by the author, there is also danger of building sand-bars across a core that is very narrow. It was found at Taylorsville that, with experienced dependable men attending the discharge, a sudden change in the flow of the water down the beach would often cause one of these bars to extend into the core from 5 to 8 ft. before much could be done to stop it. Within certain limits, the proper width of the core depends largely on the character of the material available. No core should be so wide that it would have to bear the vertical pressure of any great quantity of the material in the outer slopes of the dam, that is, the dividing line between the sand and gravel and the core should always have a rather steep slope. The writer believes that the core width decided on for the dams of the Miami Conservancy District (width of core at any point equal to height of dam above that point) was safely removed from being either too wide or too narrow and also that, in view of the limited amount of data on the behavior of core materials available at the time these dams were being constructed, it was wise not to vary appreciably from this width. However, past experience seems now to justify the conclusion that in order to accommodate different materials this rule might be varied probably

40% either way without encountering any serious difficulties and with some saving in cost.

A dry analysis of the borrow-pit material at Taylorsville showed about 40% passing the 100-mesh sieve. The material is a very compact glacial till containing no visible voids. As it was pumped into the dam it was separated, leaving the sand and gravel in the outer part and the fine particles in the core. The core was about 80 ft. wide at the base of the dam, and the large cut-off trench had to be filled with core material. This gave much more core space in proportion to the other parts of the dam than was to be had higher up. The result was that the embankment measurement for the first 15-ft. rise of the dam was a little more than 25% greater than the borrow-pit measurement. The embankment yardage continued to show an increase over the pit yardage until the dam was within about 35 ft. of the top, in spite of the fact that considerable fine clay was being wasted with the waste water from the pool. From this elevation to the top, the loss increased rapidly, and the finished embankment yardage was 12% less than the total pit yardage. After the last berms, 20 ft. below the top were passed, the dam became so narrow that the pool was nothing more than a slightly enlarged channel for the flow of the water from the discharge pipe to the overflow well and the loss from this point to the top was about 40%, that is, the embankment yardage above this point was only about 60% of the yardage of the borrow-pit required to build it. These figures indicate roughly to what extent core width may affect cost.

J. Albert Holmes,* M. Am. Soc. C. E. (by letter).—Each new contribution to the subject of earth dams is welcomed by engineers and this is especially true of Mr. Paul's paper. It is the result of investigation of materials available, methods of handling to obtain the desired results, and knowledge of previous experience in earth dam construction in all its phases. In short, the problem was approached and the work executed in full understanding and appreciation of all its important factors. Knowing the difficulties encountered and the problems involved in the making of earth dams, the writer considers the information contained in the paper to be of much value.

An interesting feature of the study is the observation of, and proof of, the escape of water from the core by upward flow in the form of small springs. The writer himself has never noted this occurrence for the reason that no clear water pools have come under his observation.

The core material in the dam at Somerset, Vt., was quite as fine as that in the Miami dams. The consolidation of the core took place during construction, as was proven by the small amount of settlement that took place after completion (0.4 ft. in about two years, with a height of 100 ft.), and the fact that no settlement could be detected during a two months' suspension of work at one-third the height of the core. Very little water escaped through the dikes except at such times as the pool was raised on the thoroughly sluiced and pervious dikes and then nothing like the quantity observed by the writer flowing from the base of the clean gravel dikes during the construction of one of the Miami dams.

* Boston, Mass.

At Somerset, colloidal clay kept the pool water always roily, and vertical springs could not have been seen if they had existed; however, after the first settlement of material to the bottom of the pool had occurred, the imprisoned water must have escaped upward as the pressure due to over-burden increased, and this in a reasonable and comparatively short time, that is, during the period of construction. That clay gives up much of its contained water under comparatively light pressures and short periods and then much more slowly, is known by experiment, and it must be assumed that whatever water remains in the core will stay there for a long time. What this quantity is, and the resulting condition of the core, is also disclosed in Mr. Paul's paper, and as he says "by far the most satisfactory and conclusive information as to the character of the cores of the Conservancy dams, has been obtained at three of the dams, by examination of the cores when they have been exposed in their undisturbed condition." This information was obtained by excavation into the core of one of the dams and, at the other two, by the loss of the pool and the exposure of the core by the washing away of the outer dike. In each case, the core resisted erosion, and what is to be more especially noted is the fact that, in one core, the water content was only 21.8 to 22.6% and, in another, about 24% by weight and this at no great depth below the top of the dam nor a long time after the material was deposited. These percentages of water indicate from 42 to 46% of voids. The conditions as to consolidation (voids) and weight (121.0 per cu. ft.) are even better than was indicated in the data obtained by Allen Hazen, M. Am. Soc. C. E., on saturated clay.*

TABLE 4.

Index number.	Percentage of water, by weight.	Specific gravity, wet.	Weight, in Pounds per Cubic Foot.	
			Wet.	Dry.
1	3.8	1.268	79.1	76.1
2	6.6	1.253	78.2	73.0
3	20.0	1.346	84.0	67.2
4	21.4	1.684	105.1	82.6
5	21.6	1.817	113.4	88.9
6	25.0	.....	.....	.....
7	25.0	.....	.....	.....
8	25.2	1.478	92.2	72.0
9	25.6	1.538	96.0	71.4
10	26.0	1.582	98.7	73.0
11	27.4	1.8±	114.0±	82.7±
12	28.6	.....	.....	.....
13	31.6	1.710	106.7	73.0
14	33.1	1.749	109.1	73.0
15	34.3	.....	.....	.....
16	38.0	1.481	93.0	.....
17	.....	1.432	89.4	.....
18	.....	1.667	104.0	.....

The writer has had only one experience in the loss of a core-pool from near the top of the dam. The material was different from that described by Mr. Paul, and there was movement for a time. The core dried out very quickly and the movement stopped. In this case, horses were driven over the core later and a trench was excavated through it by means of scrapers. The

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1724.

sides of the trench stood vertical. Some time previous to this, a sample (No. 16, Table 4) was taken from the core and found to contain 38% of water by weight.

There has been much uncertainty and speculation as to the weight per cubic foot and water content of earth in dams. Some additional data are given on this point in Table 4. It will be noted that some of the surface samples were taken during the winter months and, therefore, contained more than the average quantity of water.

No. 1.—Borrow-pit material: This is Sample No. 8 after drying under shelter in the open air for 5½ months. The sample was puddled in a box and was under no pressure. The shrinkage amounted to 2.3 per cent. There were many small holes in the mass from which the water had dried out.

No. 2.—Samples Nos. 2, 10, 13, and 14, were core material containing varying percentages of water. The specific gravity of the solid particles was 2.57. The material was part of a combined sample, made up of 266 samples taken from all parts of the core, 36% of which was finer than 0.10 mm. Voids in the dry material, 54.4 per cent.

No. 3.—Material from the down-stream slope of the dam, taken in January, and weighed in loose condition.

No. 4.—Sample No. 3 puddled in a box. After 26 days, it had settled 19.4 per cent.

No. 5.—Sample cut with 8-in. cylinder from same location as Sample No. 3.

No. 6.—From the tunnel at the base of the down-stream slope of the dam, 37 ft. in from the slope and about 20 ft. vertically, below the surface. There had been no water in the core-pool for a month previous to taking the sample.

No. 7.—From the tunnel 40 ft. above the base of the down-stream slope of the dam, 39 ft. in from the slope and about 20 ft. vertically, below the surface. No water in the pool.

No. 8.—Borrow-pit material, cut from location with 8-in. cylinder, in February.

No. 9.—Check sample from same location as Sample No. 8.

No. 10.—See Sample No. 2.

No. 11.—Somerset Dam core: Approximate weights based on 27% moisture, this percentage resulting from experimental compression of 3 tons per sq. ft.; specific gravity of solid particles, 2.62 to 2.65, and 94% finer than 0.10 mm.

No. 12.—Core material: Moisture condition on removal of core sample from test tank after having been under pressure of 3 360 lb. per sq. ft., for 11 days, followed by a seepage period of 21 days. Of a combined sample, made up of 370 samples from all parts of the dam, 39% was finer than 0.10 mm.

No. 13.—See Sample No. 2.

No. 14.—See Sample No. 2.

No. 15.—Material from the down-stream slope of the dam and sufficiently wet to move on a slope of 2.5 to 1.

No. 16.—Core material: Sample taken from 6 ft. below the bottom of the pool 48 days after water had gone from pool; surface dry; weight above sample in place about 500 lb. per sq. ft.

No. 17.—Borrow-pit material cut from location with 8-in. cylinder, in February. Compare with Samples Nos. 8 and 9.

No. 18.—Silt from river bottom-lands: This material was under the dikes, but was removed from cut-off trench in order to get at and remove the sand and gravel layer. Sample cut from location with 8-in. cylinder, in February.

In connection with the study of earth dam cores, much valuable information is given in Appendix III, "Colloidal State of Clay", of the Progress Report of the Special Committee of the Society to Codify Present Practice on the Bearing Value of Soils for Foundations, etc.*

It is there stated that, "Whatever its nature and origin, it is now fairly well established that many of the properties of clays are closely connected with the colloidal matter present, such matter being in the form of a film of colloidal gel surrounding particles which are of a non-plastic or colloidally inert nature." This colloidal material is called "ultra clay" and is present in greater quantities in new than in geologically old soil. In the presence of water, the surrounding colloidal film swells and fills, or partly fills, the voids and thus decreases the porosity. This is a part of the material called "fines" that it is desirable to waste from the core under certain conditions.

Clays are peculiarly affected by the presence or addition of various substances and differently affected by the quantity of some substances.

The writer is familiar with a test made on a clean white siliceous material that it was thought, because of its fineness, would be suitable for a dam core. Mechanical analysis showed that 70% passed a 100 and 26% passed a 200-mesh sieve. There was practically no organic matter present. The seepage rate was high, more than 500 000 gal. per acre per day.

In this material, the addition of sodium carbonate and an acid would form a gel and decrease its porosity. However, these and other substances in core materials may be one of the causes of their not consolidating.

A chemical analysis of dam cores would be quite as instructive and valuable as a mechanical analysis. Although it would not be economical to add sodium carbonate and acid to large volumes of siliceous material to make it impervious or to use great quantities of chemicals to coagulate colloids in other materials, a knowledge of substances in materials that do not consolidate, would lead to the rejection of like materials for other dams.

D. C. Henny,† M. Am. Soc. C. E. (by letter).—This paper deals, as might be inferred from the title, principally with studies of cores of hydraulic-fill dams. It refers moreover incidentally to the reasons which led to the adoption of the hydraulic method of construction. The paper states that the decision was

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 535.

† Cons. Engr., Portland, Ore.

reached only after consultation with hydraulic dam experts and after study of the mechanical analyses of the borrow-pit material. From these, it was concluded that hydraulic construction would be practicable and economical. Presumably, judgment as to the resultant quality of the finished work has also affected the decision. The practicability of hydraulicking could hardly be questioned by any one familiar with the subject and with the available material. The superior economy, however, is not clear, especially as the bulk of the material had to be loaded on cars to be handled by water as a secondary operation.

Neither is it clear that the resultant work is necessarily superior. The ideal earth dam is probably one with a tight element on the water side, heavy, well compacted, and with a slope which renders it immune from sloughing, and which is protected from wave wash, and a well drained supporting element away from the water, unyielding, stable, and of maximum weight.

Such material distributed as stated may not be necessary, but it would give the greatest factor of safety and cannot be secured by hydraulicking. The action of flowing water tends to separate sizes and to reduce specific mass gravity. In practice, also, hydraulicking has generally, as in this case, produced an open section up stream from the tight element giving access of water under practically full head to the interior of the dam and to the extent of its volume reducing the open section down stream, which is to act as support for the tight portion receiving the water pressure.

A decision in favor of hydraulicking has, heretofore, been based generally on considerations of decided economy, although sometimes the time required for construction or the character of available material may be the deciding factor. The latter, for instance, is the case with the Tieton Dam, 220 ft. high, now under construction by the U. S. Reclamation Service in the State of Washington.

In that case, material available in near-by borrow-pits consists of rock fragments in sizes from ½ cu. yd. in volume down to sand, the aggregate, after elimination of large rocks and cobbles, containing about 18% of clay. This percentage is not sufficient to render the mass tight and after slow penetration of water, the contained clay tends to lubricate the mass and thus make it liable to slide. Where springs break out, hillside slides of considerable magnitude have occurred, due partly to the cause mentioned and partly to stratification and the existence of sliding planes.

The method adopted at the Tieton Dam consists of loading cars by electric and steam shovels, with mixed borrow-pit material, hauling it to trestles, and dumping it along lines near the up-stream and the down-stream slopes. The dump is immediately attacked by powerful water jets which work it over, washing out nearly all the clay and a large part of the sand and leaving in place a ridge of rocks of all sizes with a considerable admixture of gravel and sand and a small quantity of clay. This worked-over mass is heavy and stable and stands with slopes of approximately 1¼ : 1.

Tightness is obtained by means of a concrete core-wall penetrating bed-rock and reaching to the top of the dam. The sand washed from the dumps settles in planes with a slope of about 10 : 1. The clay settles in an up-stream pool of relatively small size with a width about one-third the distance below the top of the dam. On the down-stream side of the core-wall no pool is maintained, but

the clay is continuously pumped over the core-wall into the up-stream pool, the object being to obtain sufficiency of clay on the up-stream side and to afford down-stream support for the core-wall of hydraulicked sand.

It is hoped that the author will state the reasons for adopting the hydraulic method in the case of the Miami Conservancy dams and cover this phase of the subject and also construction cost features more fully either in his closing discussion or in a separate paper.

The reading of the author's able and interesting paper brings to mind the treatment of the same subject by Allen Hazen, M. Am. Soc. C. E.,* and the voluminous discussion thereon, and invites a comparison of summaries.

There is some apparent conflict of opinion in the matter of fines which should be admitted in or excluded from the core material, Mr. Hazen contending that particles finer than 0.01 mm. may well be wasted and excluded and the author arguing that a fairly high percentage of extremely fine material in the core is not objectionable, provided the material is properly graded.

The object sought in both cases is a core which will show a satisfactory rate of consolidation and which will not remain in a liquid or potentially liquid condition for a long time.

If size of grains and their proportion were the sole elements governing drainage properties, it may be true that the presence of grains smaller than 0.01 mm. would not completely destroy favorable drainage conditions as long as the voids are not fully occupied thereby. This appears to be the case with the cores of the Miami Conservancy dams, judging from the behavior of the material. The results of physical analysis, however, do not appear conclusive in this regard, as about 30% of the material may be smaller than 0.01 mm., which might be expected to be high enough to make rapid consolidation uncertain.

The fact may well be that physical analysis in itself does not suffice to afford positive knowledge of drainage qualities. It establishes percentages between certain arbitrary limits. The gradation between these limits is not revealed nor the actual shape of the grains which may favor or hinder drainage. Moreover, when dealing with clay, colloids must have a very important bearing on the problem.

Clay has been defined† as a mixture of granular matter and a colloid gel. The importance of the colloid on plasticity is shown, which was stated to depend on a proper ratio of granular material and viscous suspension medium. Colloids or gels in clay occur as a coating of granular particles and the extent of their presence must have a decisive bearing on the facility of drainage, plasticity, and tendency to slide, yet they are not revealed by any method of physical analysis.

Thus, if certainty as to the probable behavior of clay when deposited in a supersaturated condition by hydraulicking is to be derived from laboratory tests alone, the quantity of colloidal matter must be ascertained as well as the effect upon it of excess water, both from dilution and from chemical action.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 1713.

† "The Colloid Matter of Clay and Its Measurement", *Bulletin 388*, U. S. Geological Survey, 1909, p. 59.

The favorable results obtained with the cores of the Miami Conservancy dams may be merely fortunate and not necessarily certain of duplication elsewhere under apparently identical results as to physical analysis.

That absence by wasting of grains finer than 0.01 mm. must improve drainage qualities is quite certain and the resultant material, if well graduated above this size, must be expected to be for practical purposes tight.

As long, therefore, as the effect of colloids can only be surmised and remains a matter of speculation, the wasting of fines below some limit like 0.01 mm. may be regarded as increasing safety against sliding or against the effect of much belated consolidation in a manner which cannot be definitely foretold and which may take the form of sidewise shrinkage injurious to the solidity and general tightness of the structure.

It is possible that large scale tests with the supersaturated material to be used in any given instance may throw light on the particular limit which might be adopted with safety and that this limit may be often found to be smaller than that tentatively proposed by Mr. Hazen.

A. L. Pauls, Assoc. M. Am. Soc. C. E.* (by letter).—By means of Goldbeck cells, data were obtained on the horizontal and vertical pressures in the cores of hydraulically constructed dams, that should prove valuable in the design of future dams built by this method. It is realized that only a beginning has been made, but it is hoped that experiments will be conducted on other soft core dams and the controlling factors eventually determined so that the design of a hydraulic-fill dam will be a definite scientific process in which the various factors may be correctly evaluated and the resulting structure will be the most economical type of earth dam.

The writer was Division Engineer in charge of the construction of the Germantown Dam on which the readings shown in Fig. 4 were taken. Goldbeck cells were suspended from three towers erected within the core section of the dam, one tower being situated at the up-stream edge of the core zone, another at the down-stream edge, and the third, in the center of the core zone. The readings which were taken from time to time, show continuous consolidation of the core by the gradual divergence of the lateral and vertical pressure curves. If the weight per cubic foot of the core is computed from the vertical pressure readings, shown in Fig. 4, the core, on September 10th, 1919, weighed 72 lb.; on September 25th, 1919, 88 lb.; on October 23d, 1919, 104 lb.; on November 12th, 1919, 104 lb.; on December 5th, 1919, 98 lb.; and on May 20th, 1920, 80 lb. The explanation of this variation is that winter was approaching and it was desired to cease construction operations in order to overhaul the machinery and equipment. However, before doing this, it was necessary to build the dam to such a height as to preclude any possibility of having the partly completed structure overtopped by the early spring floods. In order to attain this height as soon as possible, pumping was confined to the up-stream face of the dam and this side was built up rapidly. In doing this, the upper partly fluid portion of the core was crowded toward the down-stream face (only a few feet), which movement tilted the central tower

* Care, The Miami Conservancy District, Dayton, Ohio.

slightly. The upper part of the tower was pulled into line again, but it is likely that the movement of the core turned the cells from their horizontal and vertical positions, as they were free to move. Consequently, in all subsequent readings, the horizontal pressure would be too high and the vertical pressure would be too low. Therefore, the writer believes that the readings taken on and after December 5th, 1919, as shown in Fig. 4, are unreliable.

At Taylorsville, the installation was much better. The cells were fixed in position and registered pressure only in the horizontal and vertical planes and the consistent results obtained bear out the writer's criticism of the Germantown installation. The weights per cubic foot of the core, as computed from the vertical pressure readings, were as follows: On July 28th, 1921, 104 lb.; on October 27th, 1921, 112 lb.; and on November 26th, 1921, 107 lb.

Goldbeck cells embedded in earth, sand, and concrete, have been tested by many successive readings, and the results in each case have checked, thus showing that the theory on which the cells operate is correct. However, some skill and experience are required in taking readings. If more pressure is applied than just enough to break the electrical contact, the disk will be moved farther, and it is possible to produce a partial set in the enclosing core material, if it has reached a plastic or semi-plastic state of consolidation and thus vitiate the readings for a time.

The materials of which the Miami Conservancy Dams were built are of a glacial origin. The fines consist of powdered rock particles largely devoid of slimy colloidal clay. The absence, or rather the very small proportion, of clay undoubtedly contributed to the rapid consolidation of the cores, as shown by the author's photographs and as indicated by the pressure cell readings.

Soundings and penetration tests, with a 6-in. cast-iron ball and with iron rods, were made from time to time. These tests, particularly the soundings with the 6-in ball, showed a surprising uniformity of penetration throughout, being about 16 ft. at Germantown. These tests, together with other facts mentioned by the author, show that consolidation and drainage take place by vertical replacement. The same degree of consolidation was always found at the same depth, or in other words, the degree of consolidation of the core at any level is a function of the depth of that level. Therefore, assuming a correctly designed dam, with the proper width of core to the width of the remainder of the dam, there is no reason why construction cannot proceed as rapidly as is economical, regardless of the core, as the faster the dam is built up the faster the core consolidates.

Consolidation, probably by a process of filtration, undoubtedly takes place after the dam is finished and the core encased. This drainage, however, is slow and likely continues for an indefinite time or until the moisture accretion from the outside equals the quantity lost by drainage. Only when such a condition is reached, will consolidation cease.

Probably in no other class of construction is the quality of the finished structure so much at the mercy of the man on the job. In an hydraulic-fill dam, the mud core is "sandwiched" between two shoulders of gravel or coarse material. To maintain these distinct zones of fines and coarse material, it is necessary that they be built up together. The writer has seen photographs purporting to

show how the mud core and the gravel shoulders are built up, but rather they show how not to build an hydraulic-fill dam. These photographs usually show the mud core, drained of water, flanked by high and steep outward sloping banks, from which gravel can ravel into the core and form protruding tongues. Again, if the proportion of fines is not increased, the height of these banks will increase until a slide results, which, if large enough, may bridge the core and thus impair its purpose.

The width of core required depends on the material. The width used in the Miami Conservancy Dams was more than ample for water-tightness, besides allowing a liberal factor of safety against poor workmanship. However, for stability against the outward pressure, the core might have been somewhat wider.

GEORGE L. ALBERT,* ESQ. (by letter).—This paper is a decided contribution to engineering literature, and the data given show the value of careful designing. The studies, observations, and explorations on this work demonstrate the certainty of results that can be obtained. It would be interesting to have the subject of hydraulic-fill dams and their several modifications treated in the same able manner, for, although the core zone is one of the important elements of these dams, the other sections are of equal importance, and, in designing, it is necessary to consider the value and importance of the relation of all these sections to each other.

One of the significant features of the paper is that the mechanical analysis of core samples from the five dams shows very little difference, whereas there was a great difference in the aggregate at the borrow-pits. All the dams were built of glacial deposits, but at Taylorsville the material was glacial till, Whereas, at the other dams, it was washed-over material in the river bottoms. Special care was required in these cases to secure the results desired. These dams are good examples of what can be done by proper care and attention.

Satisfactory results can be obtained from many of the different formations that are generally termed bad, and the quality of core that can be had from a given material can be closely determined when its hydraulic characteristic is known and the method of handling is suitably selected for such material. Light, fine materials, rich in humus and colloidal matter, and apparently having no precipitating value, but controlled almost entirely by surface tension force, can be caused to consolidate by double treating or re-washing. However, there is an economical limit to such work.

The grading of core material is more or less an economical problem, and when it is necessary to make rapid progress on a fill, as stated by the author, it is desirable to have some excess of fines in the borrow-pits, so as to be able to select the quantity and quality in grading for rapid consolidation with the required impermeability. The range of grading in size between the fine and coarse materials in the core depends entirely on the hydraulic characteristic of that part of the borrow-pit materials that have to be cared for in and through the core zone, and coarse material can be admitted to a grade such as will not settle through the fines, and stratify. Core material should be confined to its zone and not allowed to build lenses of mud into the outer sections, because,

* Cons. Hydr. Engr., New York City.

permitting the core to extend beyond its theoretical limits, and the practicing of economy to the extent of trying to save everything that goes into the fill, have contributed largely to the poor record of some hydraulic fills. The hydraulic method of dam construction is important and useful, and the subject would be much benefited by more papers like that of the author.

CHARLES H. PAUL,* M. AM. SOC. C. E. (by letter).—The writer is pleased with the number and character of the discussions brought forth by his paper. The subject is one on which much more data should be assembled, and it is hoped that, during the construction of dams of this type, in the future, systematic study of hydraulic-fill cores will be undertaken and the data recorded. One of the objects of the paper was to stimulate interest along that line. Most of the discussions tend to confirm the conclusions as outlined in the Synopsis.

A study of colloids, and their effect both on water-tightness and rate of consolidation, is desirable, as suggested by Messrs. Henny, Holmes, Wiggin, and others. Presumably, a small quantity of colloidal material is a benefit, as regards water-tightness, and a large quantity is undesirable, because it may retard consolidation and decrease stability. The wasting of fines, which requires the selection of borrow-pit material having an excess of fines so that some may be wasted, the writer believes to be essential. This will automatically dispose of much of the colloidal material, if it is present, and thus permit quicker consolidation. It seems probable, however, that if a considerable quantity of colloidal material is present, enough of it would be trapped in the core to help fill the voids and thus promote water-tightness.

Mr. Dillman's discussion contains some statements which are hard to understand without further explanation as, for instance, "all central cores are a mistake * * *," and "the fineness or coarseness of the material is inconsequent * * *." He criticizes the Miami dams as being uneconomical, stating that "an inexpensive segregation of sand and gravel * * * is all that was necessary to insure safety." The writer is willing to let that question stand on its merits. Conditions in the Miami Valley required carefully designed and well built structures the safety of which could not be questioned. An inexpensive segregation of sand and gravel would not answer those requirements.

The writer agrees that a certain quantity of gravel in the core would not necessarily be detrimental. It does make a difference, however, how this gravel goes into the core, and whether or not it is well distributed. One of the great difficulties in controlling the core of a hydraulic-fill dam is to prevent masses of gravel sloughing into the core and thus substituting a porous section for what is desired to be a water-tight section.

As to performance, which Mr. Dillman suggests should be alluded to, it has not been considered that a discussion of that belongs in this paper. This also applies to wasteways. As the question has been brought up, however, it may be said that both these subjects are referred to in a paper† by the

* Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.

† See p. 1503.

writer, entitled, "Flood Control in the Miami Valley, Ohio". These dams are built for flood control only and are called into service only at times of great floods which do not occur every year. It might be of interest, however, to mention that in April, 1922, after the preparation of the paper previously mentioned, a run-off occurred in the Miami Valley, which gave a peak discharge greater than any since 1913. Without the flood-control works, this flood would have reached the old danger mark in the City of Dayton. As it was, with the flood-control works in operation, the improved river channel was not half filled. Water was backed up behind all five of the dams, the maximum depth varying from 40 ft. at Germantown to 18 ft. at Huffman. Everything worked exactly as had been expected and the public was given the first actual demonstration of how the project will operate.

Mr. Hazen thinks that the Goldbeck cells are not accurate, basing his opinion on vertical pressure readings at Germantown and Taylorsville as compared with the weight of core material from Englewood. Mr. Wiggin raises similar questions in his analysis of the pressure-cell readings, although he believes that the cells give fairly accurate readings; he thinks, however, that perhaps the pressures of Taylorsville are affected by the proximity of the concrete walls. In the discussion* of another paper, Mr. Goldbeck describes the use of his cells and their accuracy under various conditions. Some of the difficulties with the Germantown installation were described in the paper, and the discussion by Mr. Pauls gives additional information on that point. The Taylorsville readings appear to be consistent, the average unit weight of the core material, from top to bottom, being about 90% of that of the weighed sample from Englewood. As the upper part of the core is less compact and lighter than the average, so the average weight must be somewhat less than that of the lower, or most compact, part. Whether or not the cell readings are precisely accurate, they show that the lateral pressures are much less than the vertical pressures at the same depths—an indication of progressive consolidation, which was confirmed by the exposures of cores at Huffman, Lockington, and Englewood.

On page 1205, Mr. Hazen states:

"When core material is soft, the horizontal and vertical pressures are equal. As the work of construction proceeds, and more material is placed above, and as the core becomes consolidated, the horizontal and vertical pressures presumably remain more or less equal, unless something happens to disturb the equilibrium."

The writer does not agree with that conclusion. He would say, rather, that as construction proceeds, and as the bottom of the core becomes consolidated, less and less of the vertical pressure from the additional material placed above, is transmitted through the consolidated core as horizontal pressure. In other words, as the lower portions of the core lose their fluid properties, it is impossible to transmit full vertical pressures laterally. This is what would be expected, and is exactly what the pressure cells and core exposures indicate.

* "Hydraulic-Fill Dams," by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII, (1919-20), p. 1763.

Both Mr. Hazen and Mr. Wiggin refer to the term, "effective size", and would like to make comparisons of core materials on that basis. This might be of value, but it should be remembered that the term, "effective size", is an arbitrary one. It has been widely used, and is most helpful, in the matter of selection and gradation of sands for filters, and in studying the rates of percolation through them. In the case under discussion, however, a material much finer than filter sand, is being considered and not only are percolation and drainage of importance, but also stability, rates of consolidation, etc. It might be dangerous to assume that the basis of effective size would be a proper one for comparing core materials under these conditions, and for determining these other characteristics. Moreover, the true effective size of such very fine material would be exceedingly difficult to obtain.

Mr. Dunham has suggested an ingenious arrangement for securing pressure readings, and it would be interesting to see it tested in practice. The writer believes however, that it would be difficult to maintain the cylinder or shaft in place. Such difficulties have been met almost invariably in holding similar structures at other hydraulic-fill dams, and were mentioned by the writer in connection with the towers at Germantown and Englewood.

Mr. Wiggin has given some interesting analyses and has suggested a number of lines of thought which, if followed, would add valuable information to the subject. In his attempted comparisons of the Miami cores with those of the Western dams, the writer would refer him to a comparison between the mechanical analyses of the Miami cores and a typical sample from the Calaveras core, which has been published.* Bearing in mind what has been said already about effective size, it is clear that that comparison shows a marked difference in the fineness of the samples, and experience has already shown that there is a marked difference between the stability of the Miami and the Calaveras cores.

The writer believes that the penetration tests referred to by Mr. Wiggin on page 1209 are representative in both cases. He has never seen a record of penetration in the Calaveras Dam, which was not greater than that in any of the Miami dams under similar conditions.

As to whether the Miami cores would have moved if exposed at greater depth, it is to be presumed that if the depth was great enough and the face steep enough, they would have moved as any earthy material will under such circumstances. It is believed, however, that a core which is firm when exposed to a depth of 30 ft., cannot by any chance be called a fluid, and that it has consolidated sufficiently for all practical purposes. Whether or not it would flow, if exposed to some greater depth, seems to be of little consequence.

It is unfortunate that complete mechanical analyses of cores from many hydraulic-fill dams are not available. It is hoped, however, that in the future such records will be kept as will make comparisons possible. No doubt a great deal can be learned from such data.

Mr. Wiggin asks several questions about drainage which are difficult to answer. Those who have studied drainage have their own opinions as to what takes place. It seems, however, that if much of the water escapes laterally,

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1792.

it would have appeared in larger quantities than it did when pumping was discontinued. There seems to be evidence that a large quantity of the excess water is forced up vertically.

Mr. McCurdy has given a interesting discussion on some of the more important questions discussed in the paper, particularly construction problems. He emphasizes the value of rod soundings in examination of the core from day to day during construction, which probably was not given sufficient prominence by the writer. These rod soundings, as temporary records from day to day, are invaluable, and often reveal conditions which otherwise might not be determined. For permanent data, however, or for comparisons between cores at different dams, something more positive than the rod soundings is desirable. Mr. McCurdy thinks that the time element is important in the process of consolidation, and that, in a dam being built up rapidly, the core consolidates more slowly than in one on which less vertical progress is made.

Mr. Floyd, however, thinks that the weight of superimposed material is a more important element in rate of consolidation than the time element. It is probable that both he and Mr. McCurdy are correct in that both elements have their influence. Mr. Floyd gives the results of his observations on the consolidation of core under different conditions. The construction program at the Taylorsville Dam gave an unusual opportunity for such observations. His data on the swelling in volume of material from borrow-pit to embankment are also of interest. Data of that kind are desirable.

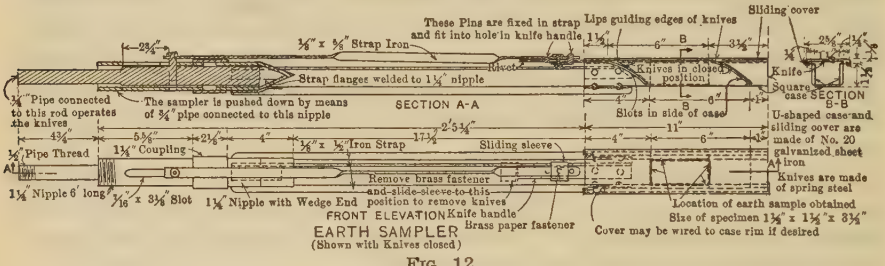
These Pins are fixed in strap and fit into hole in knife handle
Lips guiding edges of knives
Sliding cover
Strap Iron
Rivets
Strap flanges welded to 1¼" nipple
Knives in closed position
Knife
Square case
SECTION B-B
Pipe connected to this rod operates the knives
The sampler is pushed down by means of ¾" pipe connected to this nipple
SECTION A-A
Slots in side of case
U-shaped case and sliding cover are made of No. 20 galvanized sheet iron
Pipe Thread
1¼" Coupling
½" x ½" Iron Strap
Sliding sleeve
Remove brass fastener and slide sleeve to this position to remove knives
Knives are made of spring steel
1¼" Nipple 6' long
1¼" Nipple with Wedge End
FRONT ELEVATION
Knife handle
Brass paper fastener
Location of earth sample obtained
Size of specimen 1¾" x 1¾" x 3¾"
EARTH SAMPLER
(Shown with Knives closed)
Cover may be wired to case rim if desired

FIG. 12.

Mr. Justin's definitions of hydraulic fill and semi-hydraulic fill are good, and the writer believes they should be accepted as standard in future discussions of these types of construction; there has been too much confusion of those terms in the past. The definitions agree almost exactly with the descriptions of these methods of construction, under the same headings, as given in the standard specifications for the Miami Conservancy work. Mr. Justin calls attention to the desirability of having determinations of unit weight for material at various depths for use in connection with the pressure-cell readings. That would be most useful, provided there was a way of securing representative samples at different depths. The writer knows of no way in which this can be done, unless by chance there is a core exposure such as happened at Englewood and Lockington where such samples may be obtained. Ordinary boring methods are not satisfactory, because in taking the boring, the material must be disturbed, and even a slight disturbance may change its unit weight considerably. Many schemes have been pro-

posed, and some of them have been tested in the Conservancy work. One of the most satisfactory was a device (Fig. 12) evolved by Mr. G. L. Teeple, Assistant Engineer, which the writer believes comes as near to being satisfactory as anything he has seen. It, however, was not perfected in time to be of much use during the construction of these dams. It is hoped that this device, or a similar one, may be used with future constructions. Answering Mr. Justin's question relative to the method used in compacting the embankment back of the retaining wall, the writer would state that the material near the retaining wall at Taylorsville, where the pressure cells were located, was representative core material and was placed by the same methods as the remainder of the core.

Mr. Pauls has given some additional data on the behavior of the pressure cells at Germantown, and on some of the difficulties connected with installation and the obtaining of readings. He, also, believes that most of the drainage of the core takes place by vertical piping. His conclusion regarding the effect of time and rate of consolidation is that the faster the dam is built up, the faster the core consolidates, assuming, of course, proper design and suitable materials.

Mr. Henny asks for reasons why hydraulic fill was considered economical in the case of these dams. He probably has in mind a comparison with semi-hydraulic fill, or rolled fill. It is the writer's opinion that semi-hydraulic fill, as usually placed, might have been slightly cheaper than the hydraulic fill, but that the results would not have been as good; that is, in a semi-hydraulic fill, the material is not as thoroughly broken up, the core may not be as well graded, and the outside slope material is generally less pervious and not as thoroughly settled. This is usually true, unless the material is dumped from trestles and thoroughly worked over by the nozzle operating always on the end of the fill. In that case, the cost is increased, both by the trestle and the extra nozzle work, and the difference in cost between it and true hydraulic fill would be slight. At several of the Conservancy dams, large quantities of rolled fill were placed in the cross-dams which held back sections of the hydraulic fill during construction, and in some other places where, for one reason or another, that method was better suited to the construction program. In several cases, the volumes of these rolled fills were so great that a fair comparison in unit cost with hydraulic fill could be made. In every case, the unit cost of rolled fill was at least one-third more than that of the hydraulic fill. Moreover, in this part of the United States, weather conditions, have to be considered with every construction program, and it is a safe conclusion that the rolled fill would be interrupted by rainy weather which would affect either the borrow-pit or the embankment, or both, to the extent that from 20 to 30% of the working season would be lost, whereas hydraulic fill can proceed regardless of the weather. That affects economy as well as progress.

The Tieton Dam, to which Mr. Henny refers, is being constructed by the semi-hydraulic method. One of the interesting features is a concrete core-wall which is being built practically within the limits of the hydraulic core. This is the first construction of this kind of any magnitude that has come to the writer's attention, and it is hoped that a record of this work will be made

available to the Profession. The writer understands that Goldbeck pressure cells are also being installed at the Tieton Dam. The records obtained from them will be of decided interest.

As to Mr. Henny's suggestion of the usefulness of large-scale tests with supersaturated material, the writer believes that the exposures of the cores at Huffman, Lockington, and Englewood, as described in the paper, are better than any large-scale tests that could possibly be devised, and he sees no reason for wasting fines below 0.01 mm. (which, after all, is simply an arbitrary figure), when, in all five of the Conservancy dams, satisfactory consolidation has been obtained with a considerable percentage of finer material. Wasting fines is expensive, and should not be carried beyond reasonable limits. The question is, what is the reasonable limit? By going to extremes one may easily increase cost until it may be prohibitive. However, it is believed that precautions such as have been taken during the construction of the Conservancy dams are ample to insure safety, and the results seem to bear this out.

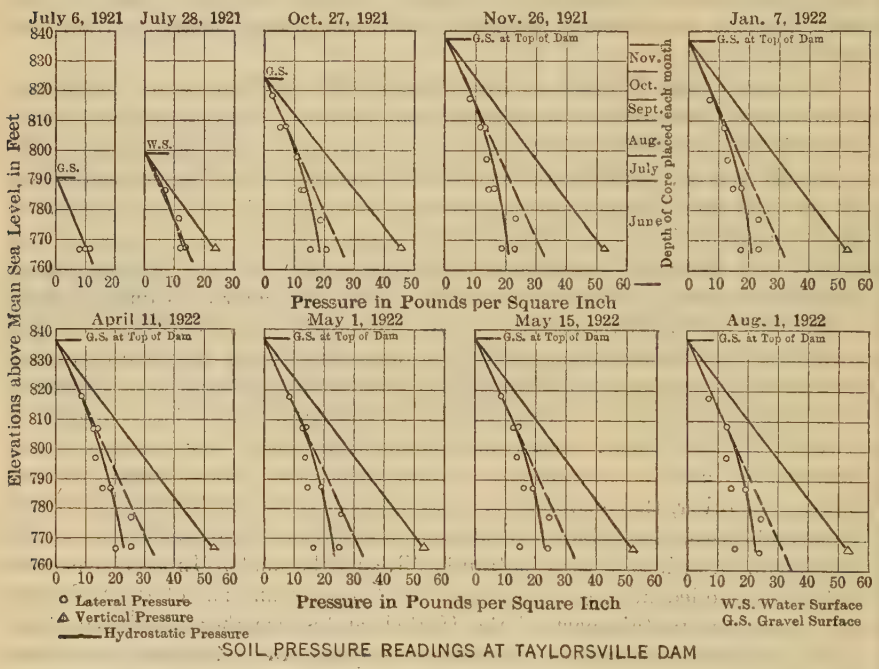

FIG. 13.

Mr. Holmes gives some interesting data regarding unit weights and water content in various core materials with which he has had experience. He does not state how the samples were obtained, but presumably they are representative of the material from which they were taken. He also suggests that a chemical analysis of core materials would be instructive and valuable. The writer agrees with him, and hopes that such a study, as well as one that will give more information about colloids, will be undertaken.

Mr. Albert's remarks emphasize again the need of care and precaution in the design and construction of a hydraulic-fill dam. His wide experience in that class of work gives additional weight to his opinions. Good results are not hard to obtain if proper knowledge and experience are applied to the problems.

Fig. 13 gives the complete results of the pressure cell readings at Taylorsville to date, including those shown in Fig. 6. It is apparent that the core had reached a state of consolidation on November 26th, 1921, beyond which the change is very slight. It may be that after construction is stopped, and no further weight is added, there will be no change in the pressure on the cells unless the equilibrium is disturbed. Very likely, the condition represented by the last five or six diagrams, where lateral pressures are from 40 to 50% of the vertical pressures, is normal for well consolidated material of that kind. It is not known whether that ratio has been determined for such materials, but 40 to 50% seems reasonable when one considers that for metals it is supposed to be about 30 per cent.

Records of settlement at the Germantown Dam are now available. As soon as the dam was finished, stakes were set along the top in such a position as not to be disturbed by frost or traffic. Measured settlement after 6 months showed a uniform relation to the height of dam at each point, and was almost exactly 0.0025% of the height in each case. After 18 months, the total settlement measured 0.0033% of the height. This also was uniform at each point. The depths of fill at these points varied from 32 to 101 ft. below the stakes.

In closing, the writer believes it will be helpful to review briefly the more important features of the subject. An editorial in *Engineering News-Record*,* by one of the Associate Editors who was on the ground at the time of the washout exposing the core at Englewood, is so pertinent to this discussion that it is quoted, in part, as follows:

"In the light of the observations on the slope washout at the Englewood dam * * * it seems necessary to revise some current notions on hydraulic dam construction. The essential question has been whether a water-deposited puddle core consolidates rapidly or remains semi-fluid for a time. Much has been said concerning this question. The discussions, with its background of the large-scale accidents at Necaxa and Calaveras, has tended to cause doubt as to the advisability of using what is certainly the cheapest, and many think the best, method of building earth dams, namely, the hydraulic fill method. But the facts of the Englewood core exposure make it plainly evident that a hydraulic core may be so built as to be solid even up to the top layers, just deposited. The section of the dam in which the washout occurred had been built up at much more rapid rate than the rest; * * * yet consolidation, or perhaps we may say precipitation, of the core silt was so prompt that the much-discussed semi-fluid zone was not 80 ft. deep nor even 20 ft., but was actually non-existent. Moreover, the core mass laid bare proved to be not merely a solid substance, but a tough substance, one possessing distinct structural value. These facts clarify effectively a field in which doubts have had a harmful influence. * * *"

The writer's conclusions as to the essential facts in the case, which he

* November 3d, 1921, p. 713.

believes are supported by the material presented in the paper and the discussions, may be summarized as follows:

1.—The gradation of core materials in a hydraulic-fill dam may be controlled within reasonable limits by proper care and attention during construction.

2.—An excess of fines in borrow-pit material is desirable, so that core control may be helped by the wasting of fines during construction.

3.—The width or thickness of the core at any elevation is an important feature of the design.

4.—The control of core width is an important feature of construction. Mr. Wiggin states (page 1210), "The well-nigh perfect control of core widths eliminated at the Miami dams the greatest element in the failure of other dams."

5.—Good results, both as to stability and as to degree and rate of consolidation, have been obtained at all five of the Conservancy dams with a high percentage of fines. With proper care and suitable materials, the same results may be secured elsewhere.

6.—Distinction should be made, in future discussions, between hydraulic and semi-hydraulic methods of construction.

7.—The practical side of the essential features of design and construction should be considered as well as the theoretical side.

8.—More data should be assembled, from future constructions of this type, as to fineness and gradation of core materials, penetration tests, pressures within the cores, and the effect of colloidal material.

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Paper No. 1503

THE AMERICAN MIXED-FLOW TURBINE AND ITS SETTING*

By Arthur T. Safford,† M. Am. Soc. C. E.,
and Edward Pierce Hamilton,‡ Esq.

With Discussion by Messrs. B. F. Groat, Charles W. Sherman, C. M. Allen, Dana M. Wood, H. A. Hageman, Robert E. Horton, Forrest Nagler, George A. Orrok, F. W. Scheidenhelm, Jay M. Whitham, Gardner S. Williams, Floyd A. Nagler, Harvey Linton, and Arthur T. Safford and Edward Pierce Hamilton.

Synopsis

The Proprietors of the Locks and Canals on Merrimack River were incorporated in 1792 for the purpose of making the stream navigable from tidewater to the New Hampshire line. The development of the water power available at the Pawtucket Falls, in Lowell, Mass., was commenced by the Proprietors in 1821, and since that time this power has been utilized in increasing amounts by the almost continuous addition of new and improved water-wheels. Much water-wheel history was enacted at Lowell during the Nineteenth Century. The late James B. Francis, Past-President, Am. Soc. C. E., was connected with the Proprietors of the Locks and Canals from November 22d, 1834, until his death, on September 18th, 1892, and was Chief Engineer from 1845 to 1885; his son, the late James Francis, M. Am. Soc. C. E., followed him in that position, until 1893. Uriah A. Boyden was closely connected with the developments at Lowell, and the first Boyden wheel was placed in the plant of the Appleton Company, in that city. Asa M. Swain was a pattern-maker in the Lowell Machine Shop and, later, developed and built the Swain wheel, at North Chelmsford,

* Presented at the meeting of May 3d, 1922.
† Engr., Proprietors of Locks and Canals; Cons. Hydr. Engr., Lowell, Mass.
‡ Milton, Mass.

Mass. The late Hiram F. Mills, Hon. M. Am. Soc. C. E. (successor to Col. Francis as Chief Engineer of the Proprietors of the Locks and Canals), first came to Lowell to test wheels for the Swain Company, and James Emerson was engaged to design a Prony brake and, subsequently, began his wheel testing there. Clemens Herschel, Past-President, Am. Soc. C. E., at various times, worked under the late James B. Francis, at Lowell, and as Hydraulic Engineer of the Holyoke Water Power Company, from 1879 to 1889, built the Holyoke Testing Flume in 1881.

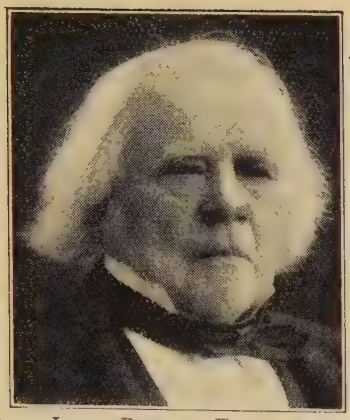
The library of the Locks and Canals contains a wealth of early water-wheel history; and in view of the present interest in high-speed and high efficiency water-wheels and their settings, particularly for hydro-electric developments, it has seemed, to the present Chief Engineer, that a review of the development of the turbine runner and of water-wheel settings may be timely. Many of the so-called modern features are from a quarter to a half century old, and were common knowledge to the men of that time. It is also hoped, by this paper, to call particular attention to the splendid work of the early hydraulic engineers and millwrights.

Such a review shows that the modern high-speed runner is the result of a gradual development, brought about by ever-increasing demands for more power, speed, and efficiency. A wheel of the high-speed propeller type was patented and on the market fifty years ago, but the mechanical connections through crown gears and belts favored wheels of slower speed, and it was not until the development of a generator of the umbrella type that the high-speed wheel came into its own.

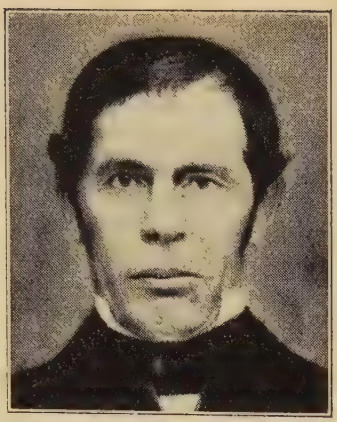
Such a review also shows that the desirability of, and the reasons for, smooth and easy passages for water-wheel channels were fully appreciated. Diffusers and draft-tubes of excellent design were in use, the scroll was well known, excellent settings were often used, and numerous examples of relatively high efficiency are on record.

During the latter part of the Nineteenth Century, quantity production with "cut and dried" installations became all too common, and too frequently the value of the proper design of the waterways and draft-tubes was overlooked. The boiler-maker ruled instead of the hydraulic engineer. Hydraulic practice during the last few years represents some improvements and a return to many of the good features developed in the middle of the last century.

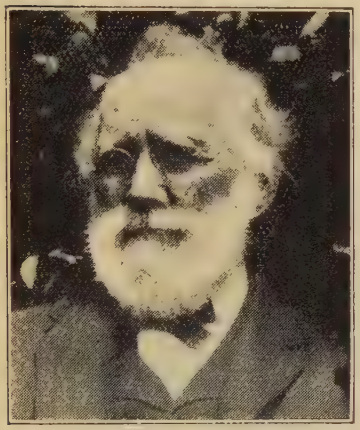
The writers have not had the benefit of the recent experimental work on wheel design, made by the water-wheel builders and their engineers, except the results of their Holyoke tests, which usually have been available. This paper does not attempt to discuss the low-speed, high-head runner, nor the remarkable development of very large units, such as some of the recent installations at Niagara Falls. In these installations, however, hydraulic conditions differ but little from those in smaller units, and the difficulties are usually mechanical and structural. It is hoped that the subject of this paper will be of sufficient interest to bring out so many additional data that the history of water-wheel design and practice in the United States may be better known than it is at the present time.



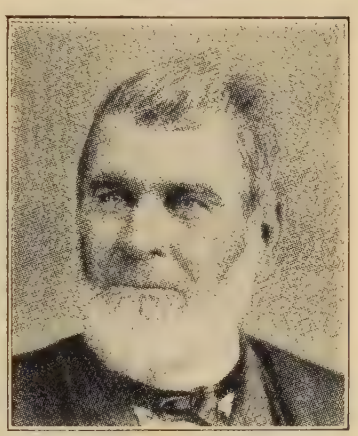
JAMES BICHENO FRANCIS.
1815–1892.



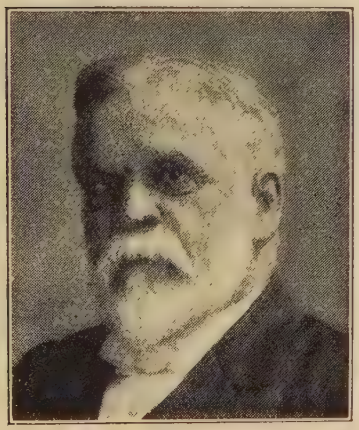
URIAH ATHERTON BOYDEN.
1804–1879.



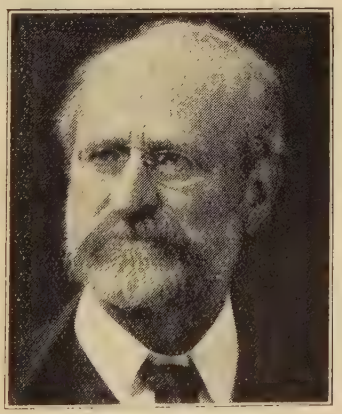
ASA METHAJER SWAIN.
1830–1908.



JAMES EMERSON.



HIRAM FRANCIS MILLS.
1836–1921.



CLEMENS HERSCHEL.
1842–

Early American Wheels

The grist-mill and the saw-mill came to this country with the first settlers. One of the very earliest water-power developments was built by Israel Stoughton, in 1634, at the Lower Falls of the Neponset River, between Milton and Dorchester, Mass., where the head was about 8 ft. In succeeding years, it was used as a grist-mill, a saw-mill, and a powder-mill, and the power is now owned by the Walter Baker Chocolate Mills. On Mill Creek, on Boston Neck, there probably were tidal mills at a somewhat earlier date, but the Neponset development is interesting in that it has been in constant use for almost 300 years. The mill moved westward with the early farmer, hard on the trail of the frontiersman. The grist-mill was a most important community center, and many a town grew up around its water power. All these early wheels must have been overshot, undershot, or breast-wheels, although there may have been flutter-wheels in later days. One may read of wheels, the buckets of which were made of ox-horns. All these old wheels were built on the spot to meet the conditions of the place.

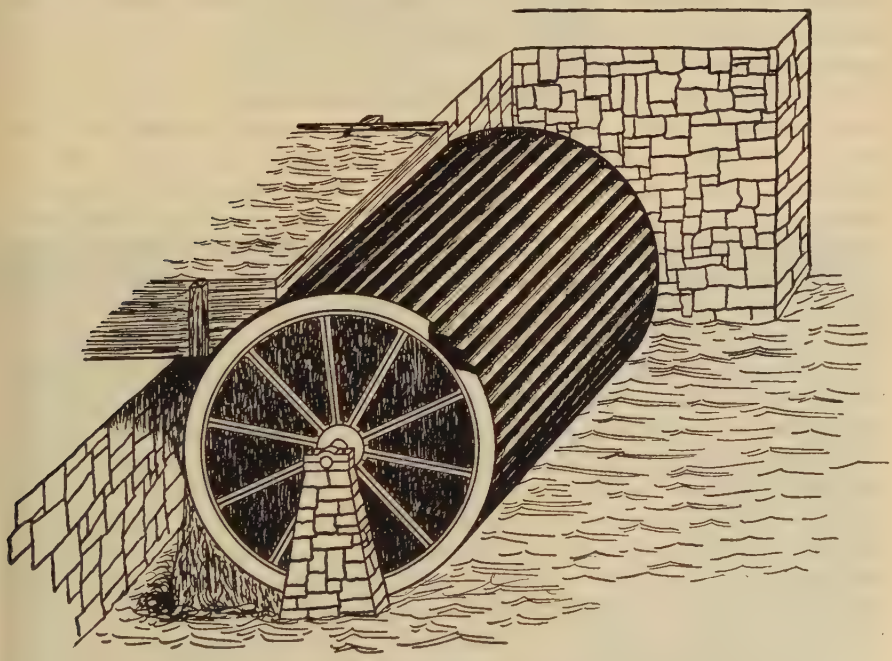


Fig. 1.—Iron Breast Wheel, Lowell, 1844.

With the coming of the Nineteenth Century, the growth of the factory system began to call for power in quantities unthought of hitherto. At first, this was met by developing the large rivers of New England on a more or less co-operative basis. The breast-wheel was used almost entirely, and as its diameter was determined by the fall—mechanical limitations fixing the former—the rivers usually were developed in stages of 10 to 20 ft. To

increase their capacity, the wheels were lengthened axially and various mechanical refinements were developed. By 1840, these wheels were the common prime movers in the big textile corporations of New England. (Fig. 1.) The example shown in Fig. 1 was one of several wheels installed in the Prescott Mills in 1844. It was constructed almost entirely of cast iron and had a diameter of 16 ft. These wheels gave good efficiencies, but were very slow, and were particularly liable to trouble from ice. They were basicly the same as those which had been used from time immemorial. As more power was required, attention was turned toward the turbine, then existing in a crude state.

There had been in use in France from very early times—Buchetti gives the dimensions of one in the Département du Gard in 1620*—an early form of turbine known as "roue à cuve", or "tub wheel", shown in Fig. 2. This was a true reaction turbine. In 1804, Benjamin Tyler, of Lebanon, N. H., patented what was known as the "Wry Fly" wheel. The wording of the patent is obscure, but it would appear that the wheel was basicly the same as its continental ancestor. The remains of an old wheel in a saw-mill, at Bow, N. H. (Fig. 3), seem to comply closely with the description of the "Wry Fly". Another early American wheel was the Parker, invented about 1828,† which was an adaptation of the Barker or Scotch Mill. It does not seem to have come into any very general use.

In the same old mill at Bow there was a pair of spiral wheels, installed on a horizontal shaft. (Fig. 4). These were reaction wheels, and because of the rather flat blade angle, gave comparatively high speed under a low head.‡ These wheels were replaced by a pair of Rose wheels (Fig. 5), which were of the impulse type, driven by a double jet. Rose wheels were common in the Northern States in the early half of the Nineteenth Century.§

In 1838, Samuel B. Howd, of Geneva, N. Y., patented an inward discharge turbine. It seems doubtful whether he appreciated the value of centripetal flow, since, in 1842, he patented an outward-flow wheel.

Such were the early American wheels. The period of the spiral and Rose wheels cannot be fixed definitely, and they may not have been as old as the others mentioned, but it is probable that they existed previous to 1840.

Transportation was very limited in the first half of the Nineteenth Century. By 1840, there were a few railroads and a considerable network of canals over a large part of the North Atlantic States. In many ways, communities, particularly those in the interior and the more inaccessible parts of the East, were largely self-sufficient, and many of the needs were met locally. The result was that the water-wheels of the period and for some time afterward were local in origin and more or less restricted to certain communities.

This is well shown by the fact that Howd, instead of trying to manufacture his wheels, licensed builders in various localities to make them in their districts. His agents sold the rights for Middlesex County, Massachusetts, to the Proprietors of the Locks and Canals on Merrimack River for about $1 200,

* J. Buchetti, "Les Moteurs Hydrauliques Actuels", Paris, 1892.
† W. C. Hughes, "The American Miller", p. 48, Phila., 1856.
‡ David Craik, "Practical American Millwright and Miller", pp. 145–149, Phila., 1870.
§ *Loc. cit.*, p. 149.

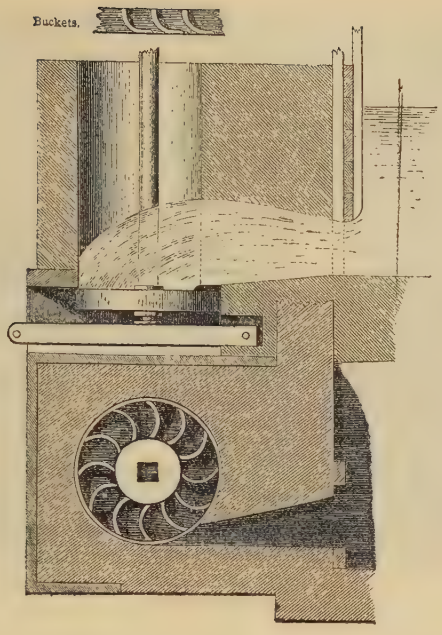


FIG. 2.—TUB WHEEL (ROUE À CUVE).

FIG. 3.—TUB WHEEL.

FIG. 4.—SPIRAL WHEEL.

FIG. 5.—FLUTTER-WHEEL AND PAIR OF ROSE WHEELS.

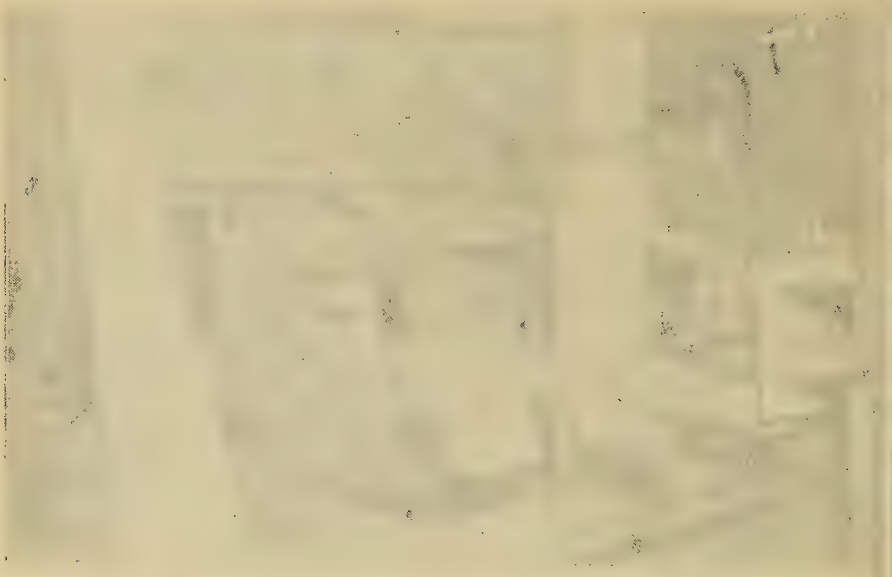


FIG. 6.—FLUTTER WHEEL.

and this corporation sold permission to use a single wheel to Solomon Dutton, of Sudbury, for $20.

By 1840, we find a few large corporations using overshot and breast wheels of considerable capacity, and a great number of small mills all over the settled districts using undershot, overshot, and breast wheels and various forms of crude turbines. (Fig. 6.) The need for power was growing, and with the increasing development of many rivers, the question of wheel efficiency was beginning to attract attention.

A country, formerly devoted to farming, and importing many of its necessities from abroad, was fast spreading westward, and manufactures were developing very rapidly to meet the growing domestic needs. As steam was still in its infancy, and water power was common throughout a large part of the country, the early factories were built where power was available. At the present time, it is interesting to study the location of the manufacturing towns of New England and to note how very many have grown up at the natural falls of the various rivers. On the large rivers, many of the developments took place where crude dams and canals had been originally built for the purpose of inland navigation.

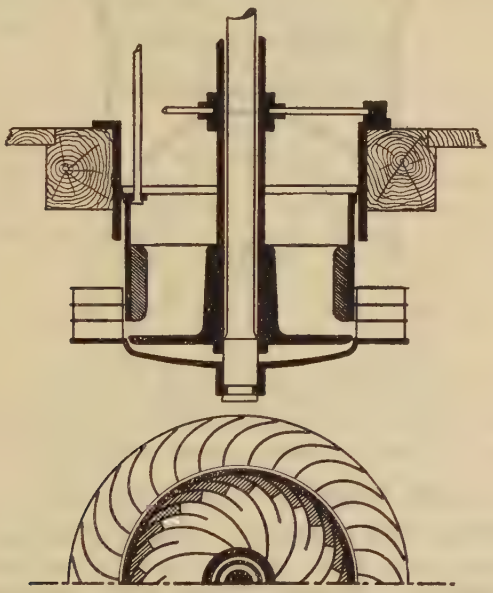


FIG. 7.—FOURNEYRON TURBINE, 1827.

THE FOURNEYRON AND JONVAL TURBINES

Fourneyron installed his first turbine at Pont sur l'Ognon (Haute Saône), in France, in 1827. (Fig. 7.) It was an outward discharge wheel in which the water was admitted axially, turned outward through 90°, and discharged. This was the first modern turbine, and it was very successful. A number were

built, including a 14-in. wheel under a 354-ft. fall, installed at St. Blasier in the Black Forest of Baden in 1837.*

The Fountaine turbine of 1839† was improved by Jonval and constructed by Koechlin, in 1841.‡ It was generally known thereafter as the Jonval (Fig. 8), and was a true axial-flow turbine, with the guides in a plane parallel to the runner. It was equipped with a straight draft-tube and was well adapted to low heads.

For many years these two turbines were the principal ones used in Europe. In the latter part of the Nineteenth Century, the inward flow turbine, as developed by Francis and Swain, was designed and built on highly scientific lines in Germany and Switzerland.

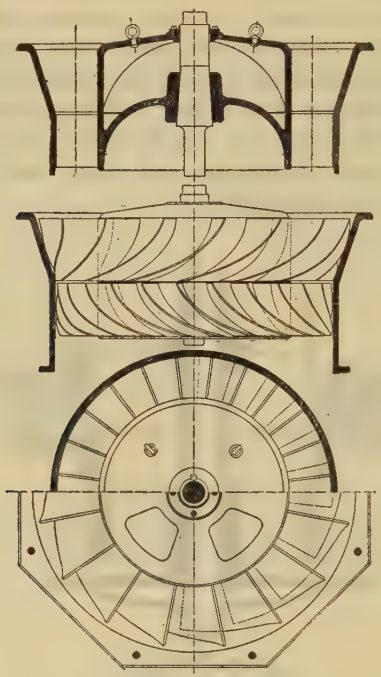


FIG. 8.—JONVAL TURBINE, 1841.

THE BOYDEN AND FRANCIS WHEELS

By about 1840, there were, in America, a number of embryonic turbines and a growing need for a prime mover of larger capacity and of better efficiency. In 1842, Ellwood Morris, in a paper published in the *Journal* of the Franklin Institute,§ described the Fourneyron turbine as then in use in France. Turbines had been mentioned in the *Journal* in 1839 and 1840, but the paper by Morris in 1842 seems to have been the first real announcement of the outward flow turbine in this century.

* Glynn, "Power of Water", p. 57, Lond., 1853.
† J. Buchetti, "Les Moteurs Hydrauliques Actuels", p. 80.
‡ J. Buchetti, "Les Moteurs Hydrauliques Actuels."
§ Vol. IV, Third Series (October, 1842), p. 217.

In 1844, Uriah A. Boyden designed, for the Appleton Company, of Lowell, a wheel along the lines of the Fourneyron turbine. (Fig. 9.) He improved and refined the foreign wheel both mechanically and hydraulically. His first wheel developed an efficiency of 78%,* and, before many years, this type of turbine was the favorite of many of the large corporations and was much used throughout New England.

In the early part of his life, Boyden was at one time a leather splitter and had a shop in Cambridgeport, Mass. He was interested in science, and in 1826, the *New Jersey Eagle* published an article by him entitled "An Attempt to Explain the Cause of the Warmth at the Poles of the Earth". In 1838, he was Engineer of the Nashua and Lowell Railroad, but after about 1840, his main work was in hydraulics. Boyden was the inventor of the hook-gauge, and he accumulated a considerable fortune from the sale of his patents on water-wheels, which, during the later years of his life, enabled him to devote his time to the study of pure science, without consideration of financial return. He was particularly interested in the velocity of light, the compressibility of water, and the study of "caloric". When he died in 1879, his fortune of about $230 000 was left to Harvard College and was used for the founding of the Harvard Observatory in Peru.

Concerning three Boyden wheels built for the Appleton Company two years later, Francis† states:

"The wooden flume, conducting the water immediately to the turbine, is in the form of an inverted truncated cone, the water being introduced into the upper part of the cone, on one side of the axis of the cone (which coincides with the axis of the turbine) in such a manner that the water, as it descends in the cone, has a gradually increasing velocity, and a spiral motion; the horizontal component of the spiral motion being in the direction of the motion of the wheel. * * * The guides, or leading curves, are not perpendicular, but a little inclined backwards from the direction of the motion of the wheel, so that the water, descending with a spiral motion, meets only the edges of the guides."

The Appleton wheels were suspended from an overhead bearing. Mr. Francis states:

"This had been previously attempted, but not with such success as to warrant its general adoption. It has been accomplished with complete success by Mr. Boyden, whose mode is to cut the upper part of the shaft into a series of necks, and to rest the projecting parts upon corresponding parts of a box. * * * It will readily be seen that a great amount of bearing surface can be easily obtained by this mode, and also, what is of equal importance, it may be near the axis. * * * The cast-iron box is suspended on gimbals, * * *." (Fig. 31.)

For a description of the diffuser which Boyden fitted to most of his wheels, Francis may again be quoted:

"The object of this extremely interesting invention, is to render useful a part of the power otherwise entirely lost, in consequence of the water leaving the wheel with a considerable velocity. It consists, essentially, of two stationary rings or discs [there was at least one example of a diffuser built integral with the runner] placed concentrically with the wheel, having an

* J. B. Francis, "Lowell Hydraulic Experiments", p. 2, N. Y., 1883.

† *Loc. cit.*, p. 3.

interior diameter a very little larger than the exterior diameter of the wheel; and an exterior diameter equal to about twice that of the wheel; the height between the discs, at their interior circumference, is a very little greater than that of the orifices in the exterior circumference of the wheel, and at the exterior circumference of the discs, the height between them is about twice as great as at the interior circumference; the form of the surfaces [of the disks] * * * is gently rounded, * * *. There is, consequently, between the two surfaces, an aperture gradually enlarging * * *. It is essential to the proper action of the diffuser, that it should be entirely under water; and the power rendered useful by it, is expended in diminishing the pressure against the water issuing from the exterior orifices of the wheel; and the effect produced, is the same as if the available fall under which the turbine is acting, is increased a certain amount. * * *

"The action of the diffuser depends upon similar principles to that of diverging conical tubes, which, when of certain proportions, it is well known, increase the discharge; * * *.

"Experiments on the same turbine, with and without a diffuser, show a gain in the coefficient of effect, due to the latter, of about 3 per cent. By the principles of living forces, and assuming that the motion of the water is free from irregularity, the gain should be about 5 per cent."

In later years, the Boyden wheels were placed at the bottom of a heavy cast-iron quarter turn. These wheels were big and heavy, and were usually set in massive cut-stone pits. They were very expensive; a 9-ft. wheel built in 1861, for the Nashua Manufacturing Company, cost $19 375.32,* or about $26 per h. p.

In 1851, Francis made a test of the power and efficiency of a Boyden wheel, built for the Tremont Mills of Lowell, which was the first scientific wheel test on a large scale. The wheel showed an efficiency of 79%; an efficiency of 88% was claimed for two of the turbines built for the Appleton Company, but the test was not so accurate. However, it is certain that the Boyden wheel, in its best form, showed an efficiency of more than 80 per cent. It had a low capacity for its size, and was of very low speed. The Tremont turbine had a specific speed of 26. The part-gate efficiency of this type of wheel was very low. The one important factor in which the Boyden wheel differed from the Fourneyron, was that its design was scientific, and every effort was made to secure hydraulic and mechanical perfection. Boyden appreciated the fact that some energy remained in the water at the time of its discharge from the runner, and to remedy this he devised the well known Boyden diffuser.

The guides and buckets of Boyden wheels were often made of composition bronze, which prolonged their lives considerably. At the plant of the Merrimack Manufacturing Company, in Lowell, there is a Boyden wheel which was installed in 1853, and which is in excellent condition and still used at certain times of the year. Sixty-nine years is a long life for a prime mover, yet this wheel, except for its low speed, is much better than many wheels of much more modern design now in use in New England and elsewhere.

From a consideration of the Boyden wheel and of the earlier Howd wheel, Francis became interested in the inward flow turbine. The Proprietors of Locks and Canals, of which Company he was Engineer, secured the patent

* J. B. Francis, Papers, Vol. 18.

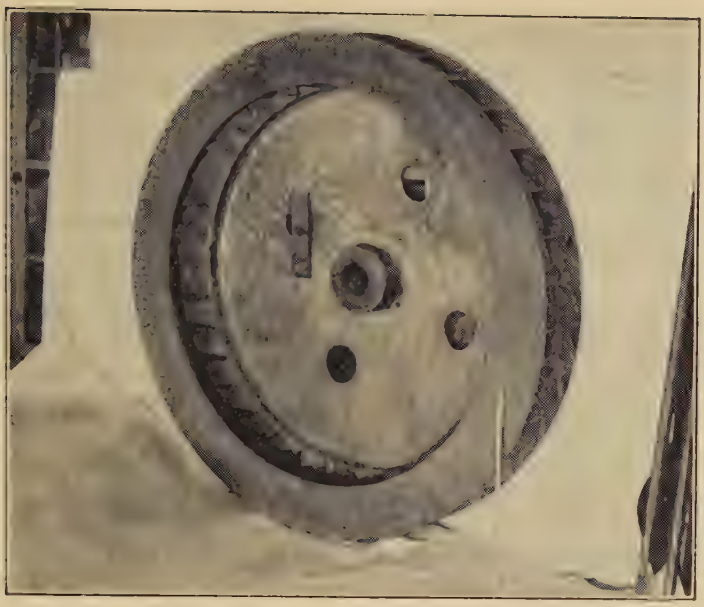


Fig. 9.—Boyden's First Turbine, 1844.

Fig. 10.—Francis' First Experimental Wheel, 1847.

rights for the Howd wheel for the locality in which Lowell was situated. Francis first built and experimented with a small model wheel which developed an efficiency of 71% and a specific speed of 19. (Fig. 10.) In 1849, he designed a center-vent wheel for the Boott Mills, which gave an efficiency of almost 80 per cent. A wheel of this type, installed in 1847, is still used to operate the head-gate hoist in the Northern Canal Gate-House in Lowell, and there is another (Fig. 11) in the grist-mill belonging to the Proprietors. This wheel, although theoretically better than the Boyden, was a true inward discharge wheel, and, as such, was restricted in power. In its original form, it was little used in the United States and soon disappeared, but it was the foundation on which the modern American water-wheel was built.

These two American wheels—the Boyden and Francis—marked a great step forward. They were designed scientifically, full-sized drawings were made of the guides and buckets, and the path of a particle of water was studied for different relative velocities under a given head. Professor W. P. Trowbridge states that all the principles on which wheels had been designed, previous to this period, were wrong,* and that Boyden and Francis broke away from tradition and established entirely new methods. De Volson Wood† made complete computations for the original Francis wheel (Boott) and came to the conclusion that the design was excellent. The computed efficiency was 79.31% and the observed efficiency, 79.37 per cent. The result of this period of scientific design has been the production of one thoroughly American wheel, the Howd wheel, as developed by Francis. This wheel was to be the forerunner of all modern reaction wheels, and the next step will be to trace its development into the mixed-flow turbine.

The "Cut and Try" Period

The period after 1860 was marked by the development of a great number of wheels of all types and combinations of types, good, bad, and indifferent, but mostly very bad. The sentiment of the period has been well expressed by "W. W. T.",‡ as follows:

"There is no use denying that our object was to make money. We had seen these parties build an ordinary casting weighing about 300 lb. and worth when finished $30, and charge for it $231. Such profits as that were well worth working for, so we made the experiment."

It happened that, obtaining an efficiency of only 51%, they were honest enough to give up the attempt, but other makers, turning out even worse wheels, were less scrupulous, and continued to foist their wheels on a misguided public.

Another remark typical of the period is one attributed to Swain, inventor of the wheel of that name. Before a test of one of his wheels at Lowell, when some one said that the wheel bound in its bearings, he "guessed it would go, only put the water to it". It did not take many years of testing at Holyoke to banish that idea, and nowadays some manufacturers even have special ball-bearing bridge trees made for their Holyoke tests.

* "Turbine Wheels", N. Y., 1890.
† "Turbines", *Transactions*, Am. Soc. Mech. Engrs., Vol. XVI.
‡ *Emerson's Turbine Reporter*, January, 1876.

Crude and unscientific as this period was in many ways, it gave, as its result, the modern American mixed-flow turbine, and it is well worth while to examine closely the various elements that went into the melting-pot. Since this period, almost every wheel has been a combination of the tub wheel in its many forms, the center-vent Howd-Francis, and the axial-flow Jonval. It would seem, perhaps, that a little too much credit was given to the Jonval, since almost every wheel, except the true Jonval, is merely a combination in some form or other of the first two named. All the scientific teachings of Boyden and Francis were thrown to the winds, and the great god, "Cut and Try", came into his own. If a wheel did not come up to expectations, its buckets were chipped back, up, or down, or its blades pounded, until it gave something better. Such a period could hardly be avoided, since mathematical analysis and design of turbines were unknown to the majority of early wheel makers. The beginning of the testing system at Lowell, and, later, at Holyoke, did much to relieve the situation. Before many years, a manufacturer could not avoid a wheel test and sell wheels, with the result that a poor wheel was either improved or abandoned. For a long time, a few makers managed to avoid public tests, but gradually they were forced to make such tests, and by 1890 most of the wheels on the market were more or less satisfactory. During this period, combinations of all kinds were tried, and great ingenuity was shown, with the result that, by 1873, reported efficiencies of 90% had been reached.

One of the early wheels was the Warren scroll, which dated from about 1853. It was a slightly modified Francis runner placed in a wooden scroll case without any guides. It was merely an offshoot of the Howd-Francis wheel and, except for its casing, was of little importance.

In 1858, Swain made his first 6-in. model. This wheel was much like the Howd-Francis, except that its buckets were deeper, many in number, and curved outward from the inner discharge edge, so that the discharge was inward and downward. The diameter of the Swain wheel was still large for its capacity, and its buckets were rather shallow, but it was a really good wheel and was the direct predecessor of the modern low-speed reaction wheel. In 1870, on the advice of the late Mr. Mills, the bucket entrances were deepened and the capacity was somewhat increased. A test on a 72-in. Swain wheel by Francis, at the Boott Mills, in 1874, showed an efficiency of almost 84%, with a specific speed of 40. At this time, the wheel had no draft-tube. In 1909, C. M. Allen, M. Am. Soc. C. E., tested the power of this wheel, after a draft-tube and bevel crown gears had been added, and the efficiency, measured by an Alden dynamometer on the jack-shaft and by current meter, was 86.1 per cent.

A power test made in July, 1921, in which a small generator was driven by a belt from the jack-shaft, showed an over-all efficiency, at the switchboard, of 73.9 per cent. After almost fifty years of use, this wheel seems still to be as good as ever; its bronze buckets are in almost perfect condition, and, to all appearances, it is capable of running for another fifty years. It is set in a wooden flume, and the water enters eccentrically, as in the Appleton wheel previously mentioned.

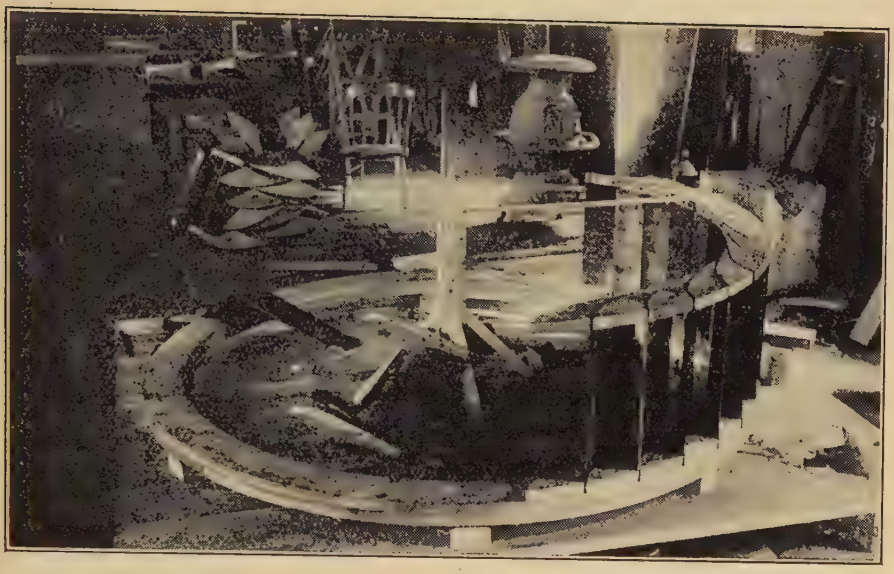


FIG. 11.—FRANCIS WHEEL IN LOCKS AND CANALS GRIST-MILL.

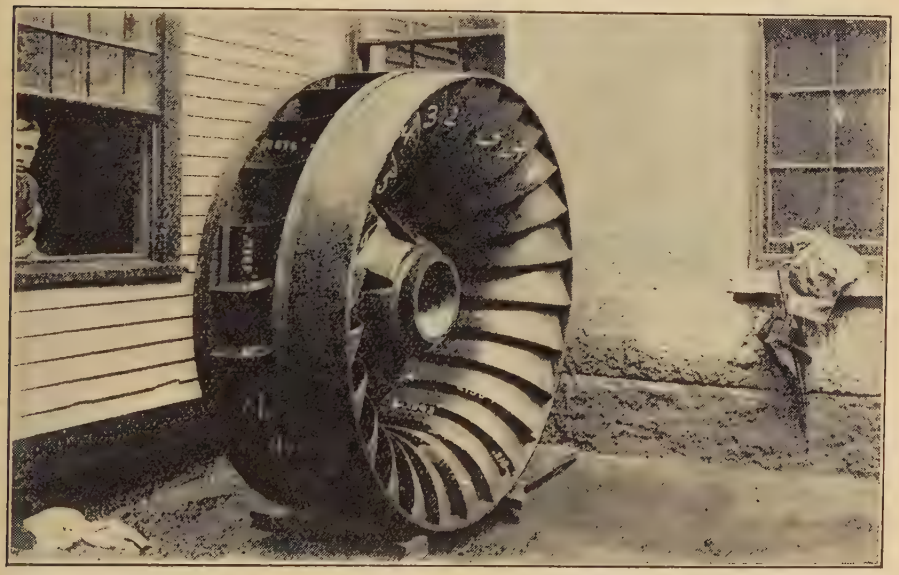


FIG. 12.—SWAIN RUNNER.

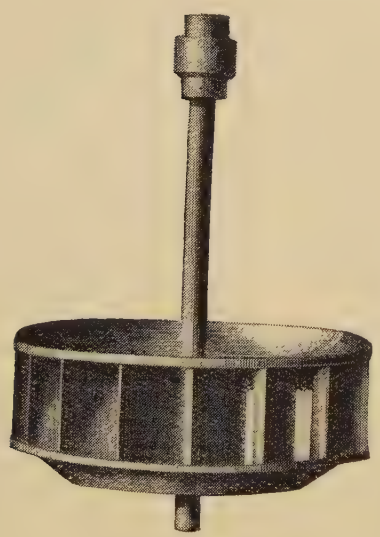


FIG. 13.—RUNNER OF "AMERICAN" TURBINE, 1859.

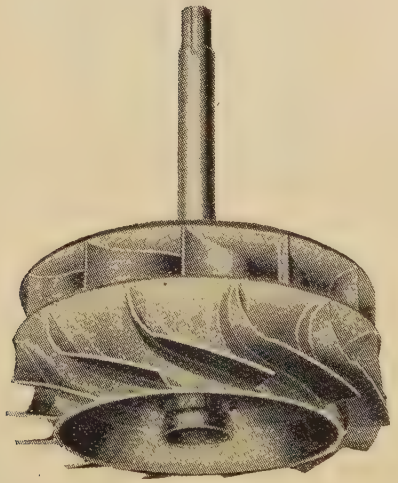


FIG. 14.—RUNNER OF LEFFEL TURBINE, 1862.

The Swain wheel, although one of the first of this period, was modern in every way as far as the runner was concerned. (Fig. 12.) The gate was cylinder and rather clumsy, but the part-gate efficiency was very high for that time, perhaps because the gate opened downward rather than upward. This wheel was a direct development of the Francis runner, probably by "cut and try" methods, as the inventor was a pattern-maker. However, it was really an excellent wheel and was the father of the modern wheel, just as the Howd-Francis was the grandfather. About 1871, a number of these wheels were installed in Lowell, replacing Boyden, center-vent, and breast-wheels, and several are still in use. In all the Swain wheels, great attention was given to the smoothness of the water passages, and hydraulically they were nearly perfect. The lower step bearing, shaped as an inverted cone, diverted the downward discharge to a horizontal direction with a smooth and easy transition.

The "American" water-wheel was patented in February, 1859. It was a Francis runner, modified to give an inward and downward discharge. Its buckets were shallow, and the size was still large for the capacity. (Fig. 13.) This wheel had attained a wide distribution over the United States by 1870, particularly in small saw-mills and grist-mills. It was the first stock wheel and marked the beginning of the period of quantity production of standardized wheels. As wheel capacities were increased, the "American" was developed, through various stages, to the "Improved New American" wheel, as made in recent years by the Globe Iron Works, of Dayton, Ohio, successors to the original firm of Stout, Mills, and Temple. The first "American" wheel, as tested by Emerson in 1872, showed an efficiency of about 80% with a specific speed of 25.

The firm of James Leffel and Company began building wheels at Springfield, Ohio, in 1862, and has continued to do so to the present time. The first Leffel wheel was double; that is, it combined an inward discharge Francis runner with another of the inward and downward type, built together in one solid piece. (Fig. 14). This wheel attained even greater distribution than its contemporary, the "American". Equipped with wicket gates, patented by Elijah Roberts, of Rochester, N. H., in 1854,* it gave an efficiency of about 74% and a specific speed of 30.

Many manufacturers built the double wheel during this period, but all of them soon died out, except the Leffel. It is understood that the makers say that they do not quite know why the double wheel works, but it does, as shown by the fact that the double discharge Leffel Type F has recently shown an efficiency of 93% at Holyoke. It is rather interesting to note that the Leffel Company still sells a considerable number of its original model, although it makes several types of wheels, including one of a specific speed of 102.

The Houston wheel had a runner resembling somewhat a bevel gear, the inlet edges of the buckets being at an angle of about 45° with the shaft, the bottom diameter being the greater. This wheel was one of the better turbines of this period, showing an efficiency of as much as 88%, with a specific speed of 32. It did not have much to do with the development of the modern

* *Emerson's Turbine Reporter*, January, 1876.

wheel and gradually disappeared; however, some modern high-speed wheels, especially a recent continental one, show a return to this shape of tilted bucket.

The Risdon wheel (Fig. 15) which attained an efficiency of more than 90% in 1873, marked the high point of the slow-speed, low-capacity period. Despite the fact that it had a cylinder gate, it showed an efficiency of 73% at half load, the highest attained so far. It was an inward and downward flow wheel, and differed from the Swain wheel mainly in that its discharge was wholly axial. The Risdon Company was the first, since Boyden, to appreciate the diffuser or draft-tube, and to apply it to its wheels. The wheel tested in 1873 gave an efficiency of 90.5% with the diffuser, and 88.8% without it. The hydraulic design of the Risdon wheel was excellent, and it seems to have been satisfactory mechanically, although, at one time, some trouble was had with the gate mechanism.

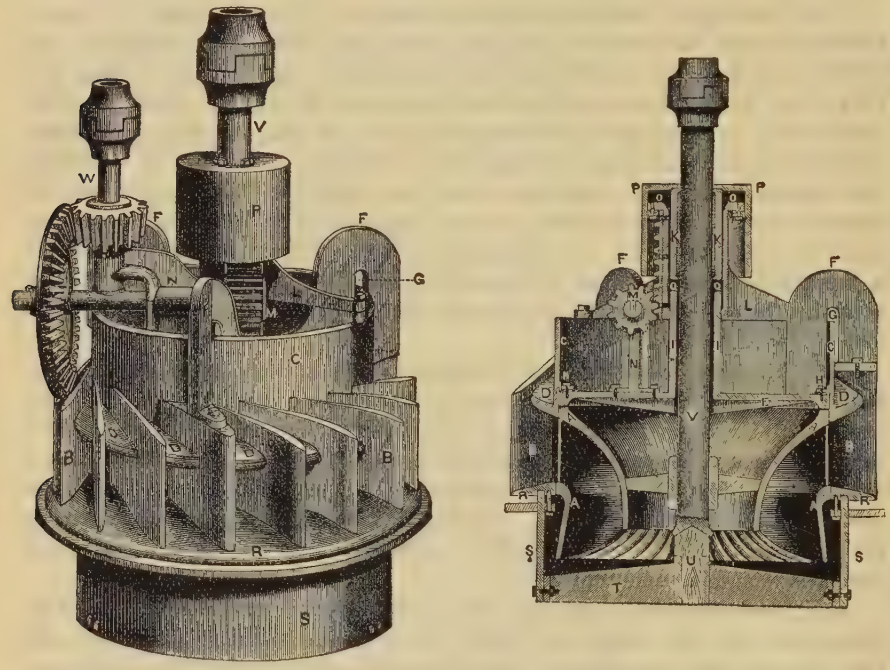


FIG. 15.—RISDON WHEEL, 1873.

Many pages would be required to describe some of the other wheels of the period such as the Angell, Barber, Blackstone, Bodine, Case, Curtis, Cook, Geyelin, Humming Bird, Humphrey, Luther Scroll, Tyler, Upham, Whitney, and others. However, they had little to do with the further development of the turbine, and, with the coming of the later wheels, soon disappeared. An especially interesting wheel of this time was the Wynkoop, a combination of an impulse wheel discharging into a reaction wheel. Its conservative makers claimed a mere efficiency of 175%, but on test it showed about 55 per cent. At this time, the idea was prevalent that a double wheel would give very high

efficiencies, and many of the ideas conceived bordered on perpetual motion. Of all the combination wheels, the Leffel wheel was the only one which continued beyond the end of this period.

The Propeller Type of Runner

Recently, there has been placed on the market a propeller type of runner, for which many advantages are claimed, chief of which is a high specific speed. It is proposed to demonstrate again the truth of the old adage, "there is nothing new under the sun", and to show that wheels of almost exactly the same type were used many years ago, and that their high speed was appreciated. It is claimed that the modern propeller wheel is axial flow. Exception is taken to this, as it is difficult to understand how, in the matter of direction of flow, the propeller wheel can be different from any modern high-speed reaction wheel. The propeller wheel, as commercially installed, is set in a wheel case of the Francis type, the water being admitted through the usual wicket gates, in a direction approaching the tangential.

True axial flow presupposes axial delivery of the water to the casing, as in the Jonval wheel. The propeller wheel must depend on the principle of the vortex, which is formed by the tangential admission of the rapidly moving water. It is only a step farther to imagine a wheel setting in which, guides being eliminated, a scroll may be substituted, of such dimensions as to give the same direction and velocity of flow to the wheel. It is granted that this will hold under only one condition of speed and discharge. The wheel is still a propeller wheel, and it is still operating on the same principle as it did when equipped with wicket guides.

The old tub wheel consisted merely of a vertical shaft carrying a number of blades lying in planes inclined to the wheel axis. The water was admitted to the wheel with a more or less downward direction, and eccentric to the axis. Under such conditions, somewhat of a vortex action must have resulted. This type of tub wheel was a true reaction turbine. How, then, but for the addition of the draft-tube, is the modern propeller wheel different? In both cases, the water is admitted practically perpendicular to the shaft, and discharged axially, and the runner is of the same type.

In July, 1860, J. W. Truax, of Richford, Vt., patented a water-wheel known as the "Green Mountain", which, as manufactured in 1876, was as shown in Fig. 16. It was a four-bladed wheel on a vertical shaft, set eccentrically in a wooden flume. The runner had a band around it, to which were attached a number of small, inclined plane saw-teeth, designed to make the leakage do work, but this does not change the fact that here is a runner identical in principle with the propeller wheel as made at the present time. Other than the maker's circular, little information can be found regarding this wheel. Apparently, its use was largely local, most of the known installations having been in Vermont. Truax, in his circular of March, 1876, states:

"What has baffled the skill and ingenuity of inventors, has been to produce a wheel that would use a large quantity of water and obtain corresponding power for the water used, and, at the same time, attain sufficient speed in the wheel itself to allow the power of the wheel to be transmitted with convenience and small outlay on light falls. These points are obtained with this wheel."

He further states that the speed of a wheel of this type depended on the incline of the blades. Not only did Truax appreciate what he had, but he also knew how he got it, which was unusual at this time.

Unfortunately, no tests of this wheel are available, and the only record of it is the maker's wheel-table, which is claimed to be the result of actual measurements. It gives to a 30-in. wheel a speed of 126 rev. per min. under a head of 1 ft. Assuming that the measurement of the revolutions per minute and of the water was correct, and, furthermore, assuming an efficiency of 60%, which seems fair and reasonable in the light of the performance of similar wheels of the same period, a specific speed of 125 is obtained. If the efficiency had been only 40%, the specific speed would still be more than 100.

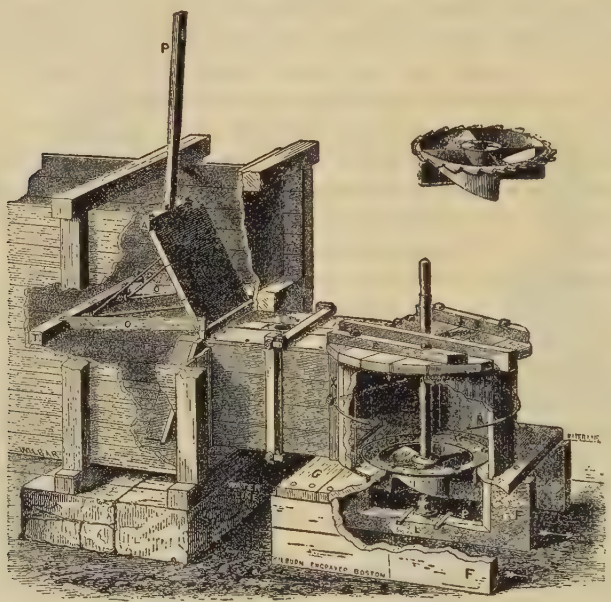


FIG. 16.—GREEN MOUNTAIN PROPELLER, 1876.

In June, 1884, at the Holyoke Water Power Company's flume, a test (No. 256) was made of a Chase Special wheel (Fig. 17), built by the Chase Turbine Manufacturing Company, of Orange, Mass. This wheel had an eight-bladed propeller runner, set, without any guides, in an iron scroll case. Here is another wheel, somewhat more modern, operating on the same principle. The buckets were less flat than in the "Green Mountain", with a consequent reduction in speed. However, it showed a good efficiency, 78.9%, with a specific speed of about 50.

In the Jonval, the blades were set radially around the periphery of a disk, which left a circle of dead area around the shaft. This meant that the effective lever arm of the runner was large, with a result that the speed was necessarily low. Both the Chase wheel and the "Green Mountain" differed from the Jonval, in that their blades were directly attached to a small hub,

with almost no dead area. The blades were effectively acted on by the water to a point very close to the wheel shaft, which meant that, for a given diameter of wheel, a much higher speed could be attained.

With regard to "spiral, or screw flood wheels", David Craik,* in 1870, stated:

"Their principle of action is the same as the screw propeller, which has, in a measure, superseded the paddle-wheel in steamboats—the difference being that the propeller is driven round * * * by the steam-engine, * * * while the screw water wheel * * * is driven or revolved by the force of the passing current against its oblique vanes * * *. To comprehend this similarity better, take a screw propeller, and place its axis upon suitable bearings, and parallel with the stream in a strong, uninterrupted current, and entirely submerged, and it will furnish a motive-power * * *."

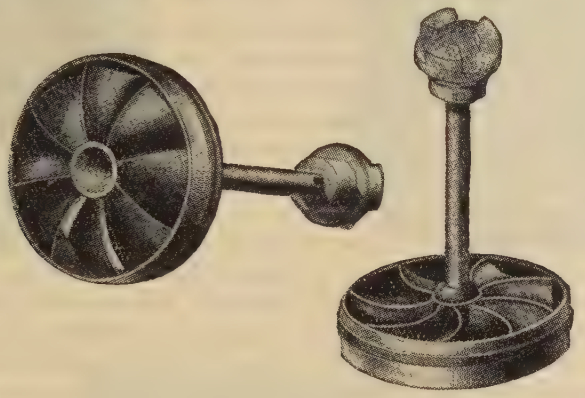


FIG. 17.—CHASE SPECIAL RUNNER, 1880.

He further states that, by this, he does not advocate using the common type of propeller for a wheel, as being driven rather than driving, "it requires a modification in structure and details". This refers, of course, to a wheel with true axial flow, and one in which the water approaches without any whirling motion.

R. E. Horton, M. Am. Soc. C. E., states:†

"A variation of the Jonval turbine, in which the number of buckets was reduced to two, was extensively used in saw-mills in northern New York. Owing to the large openings of the buckets, ice, drift, and other obstructions could pass through this wheel without injuring it. The vanes were nearly horizontal, giving a high speed of rotation. The efficiency was very low."

This was the Austin wheel, which was similar to the Truax wheel.

THE MODERN MIXED-FLOW TURBINE

In 1876, a number of 24-in. wheels, invented by John B. McCormick, of Brookville, Pa., were sent to Holyoke to be tested. They were the development of a type of large capacity wheel, invented by John and Matthew Obenchain, and were the first of the famous "Hercules" type. One of them, tested by Emerson, showed an efficiency of 89.2% with a specific speed of

* "The Practical American Millwright and Miller", p. 150, Phila., 1870.

† U. S. Geological Survey, *Water Supply Paper No. 180*, p. 13.

48, which is 13 more than the highest previous wheel, the Risdon. A 36-in. "Hercules", tested at the present Holyoke flume in 1883, showed an efficiency of 87% in three different tests. The flow was inward, downward, and slightly outward. The buckets were much deeper than in any previous wheel and protruded below the band, thus allowing the outward discharge. (Fig. 18.) The wheel was of the cylinder-gate type and the blades had fins parallel to the line of flow, that were supposed to improve the part-gate efficiency which was 73% at half power.

The production of the "Hercules" ushered in a new period of wheel design. The inward flow principle of the Howd-Francis and the downward discharge of the tub and Jonval wheels had been combined in the correct proportions, and, aided by good mechanical construction, there had been evolved the American mixed-flow turbine. High speed was yet to be developed, but the beginning had been made, and, with few exceptions, all subsequent design was merely improvement over the McCormick type.

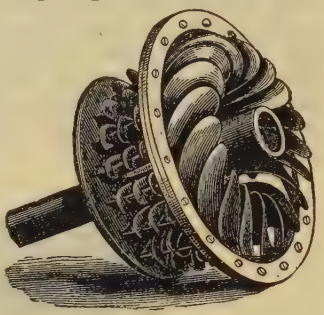


FIG. 18.—HERCULES RUNNER, 1876.

One must not forget the generations of millwrights who had worked to make this possible. It was not a sudden invention; it was merely the crystallization and modification of principles toward which all had been working. Many an old, self-trained mechanic contributed his mite to the development of this new type. This was a real American production, the result of evolution during a changing period in American history. The need arose, made itself felt, and eventually was met, not by the work of one great scientist, but by the multitudinous efforts of an army of old Yankee millwrights and machinists, many of the names of whom are either unknown or forgotten.

The year after the introduction of the "Hercules", Stilwell and Bierce, of Dayton, Ohio, placed the "Victor" turbine on the market. This wheel was much like the "Hercules" and had very deep buckets which hung even farther below the band than in the earlier wheel. Built with a register gate, it was of very simple and solid construction, and, although a little below the "Hercules" in efficiency, it also had a specific speed of 48. (Fig. 19.)

Both the "Hercules" and the "Victor" were later modified somewhat; the "McCormick" wheel was placed on the market by several makers, and the improved Leffel, double-discharge wheel, the "Samson", appeared. Until about 1900, these were the usual types of wheel and practically every condition necessarily was met by the selection of some one of the stock models of the several types then available. Such was the state of affairs at the beginning of the period of electrification, when higher speed and better regulation became of growing importance.

During the last few years, all effort has been devoted toward increasing the specific speed of the mixed-flow runner, without suffering a loss in efficiency. The bottom, or discharge, diameter has been greatly increased, with

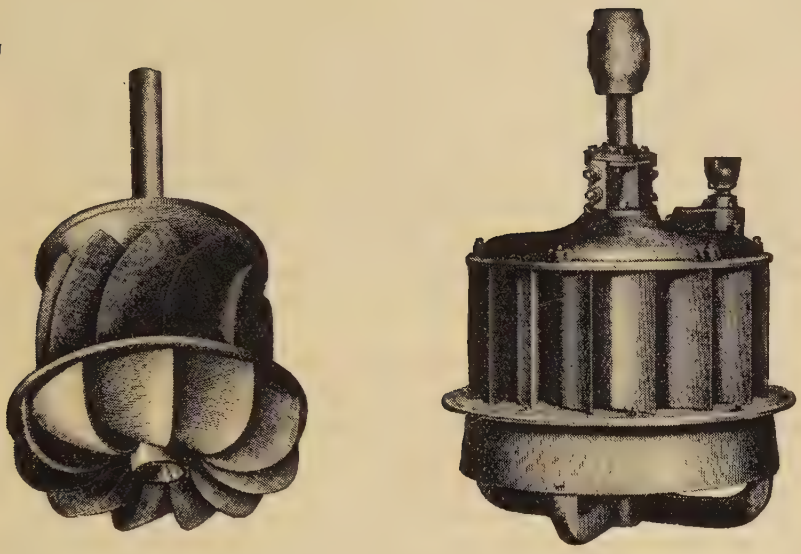


Fig. 19.—Victor Turbine, 1877.

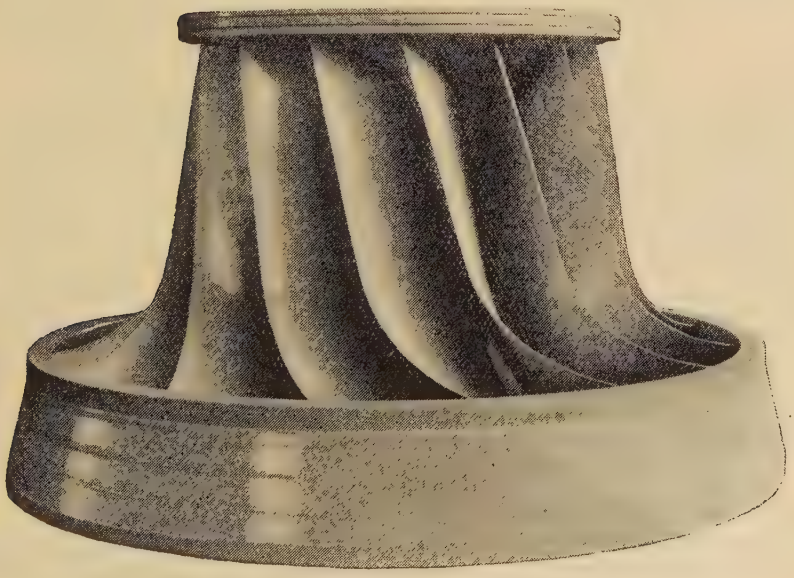


Fig. 20.—Modern High-Speed Runner.

the result that the modern high specific speed runner with a nominal diameter of 30 in., has a maximum diameter of more than 40 in. The buckets no longer protrude much below the band, and the inlet edges of the buckets usually slant, so that the clearance between the guides and the buckets is considerably more at the top than at the bottom. Fig. 20 shows one of the best of the modern high-speed runners, which gives an efficiency of more than 90%, and a specific speed of 102.

In the United States, there seems to be a general belief, especially among some of the large water-wheel manufacturers, that practically all the development of the modern water-wheel has taken place within the last twenty years and has been the result of the introduction of foreign practice in water-wheel design. Practically all development abroad has been in Switzerland and in Germany. In France, where the first turbines of record are found, comparatively little has been done toward the development of the modern high-speed wheel.

The Germans developed the mixed-flow turbine to some extent, but they did not create it—that honor belongs to the United States. The turbines of Fourneyron, Fountaine, and Jonval were all French in conception, although it has been said that Henschel, of Cassel, Germany, deserves the credit for the axial flow wheel, rather than Jonval, whom he may have preceded by a year or two. The Fourneyron and the Jonval were, for a long time, the only turbines used in Continental Europe. Meanwhile, the idea was exported to America, where Howd and Francis developed the forerunner of the mixed-flow turbine. This, in turn, was sent back to Germany and Europe took up the inward flow principle at about the time the development of the "Hercules" introduced a new period of wheel design in this country.

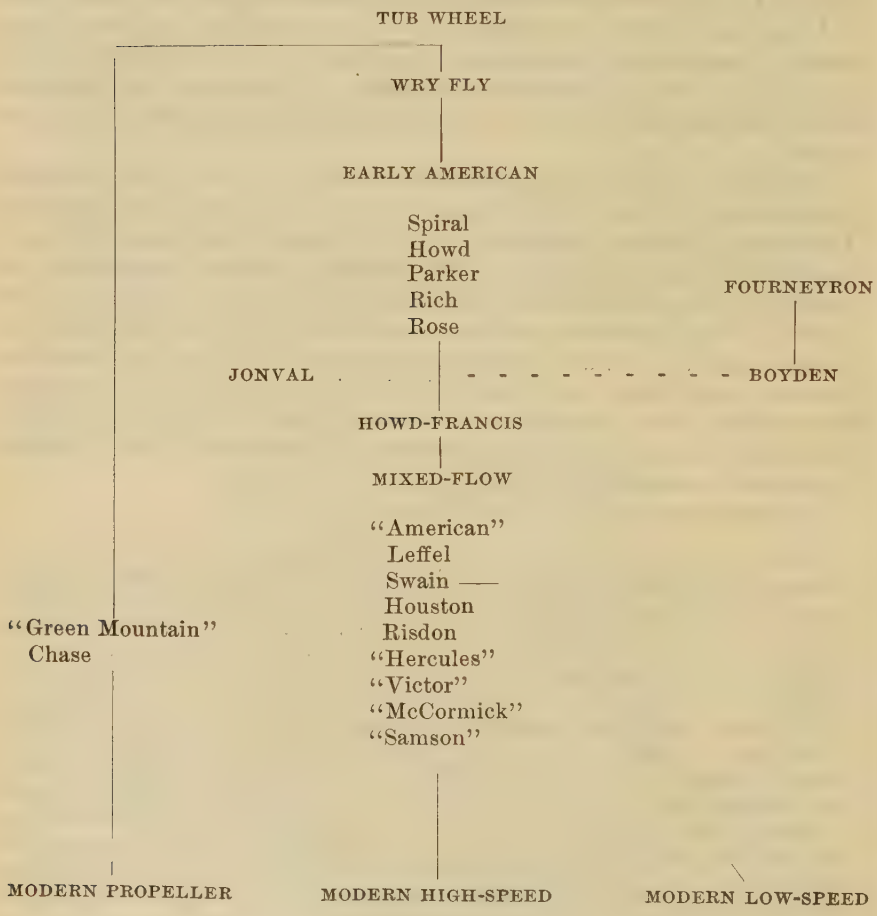
A Swiss engineer, Mr. A. Streiff,* states:

"It is a matter of fact that every marked improvement in the design of pressure-type wheels originated in America. The classic investigations by James B. Francis of his 'center-vent' water-wheel at the Boott Cotton Mills at Lowell in the year 1850, are the foundations of the modern Francis turbine. Professor F. Prasil, of Zürich, in his theoretical investigations published in the year 1905, did not hesitate to illustrate his conclusions with examples taken from the original Lowell experiments. It is, perhaps, not to be regretted that the Francis turbine was not reared in the same scientific atmosphere in which it was born, since this might have stifled the innumerable original creations of the inventors who followed. Each small foundry, and machine-shop, so to speak, manufactured water-wheels based on Francis' principle, and hardly any new form of runner can be conceived that has not been made in the past, and is perhaps still running in some New England mill. It was not until the year 1875 that the J. M. Voith Company, of Heidenheim, Germany, took up the Francis wheel, and in 1876 that the Escher Wyss Company, of Zürich, Switzerland, followed suit."

Mr. Streiff further states that, in 1914, American practice in the construction of high-speed, low-head runners far surpassed the best that could be produced in Europe. Table 1 gives the characteristics of American water-wheels, for the period 1847-1900, on the basis of a 30-in. wheel.

* "Electrical Engineering and Hydro-Electric Development", *Transactions*, Inter. Eng. Cong., 1915, p. 498.

FAMILY TREE OF MIXED-FLOW TURBINE



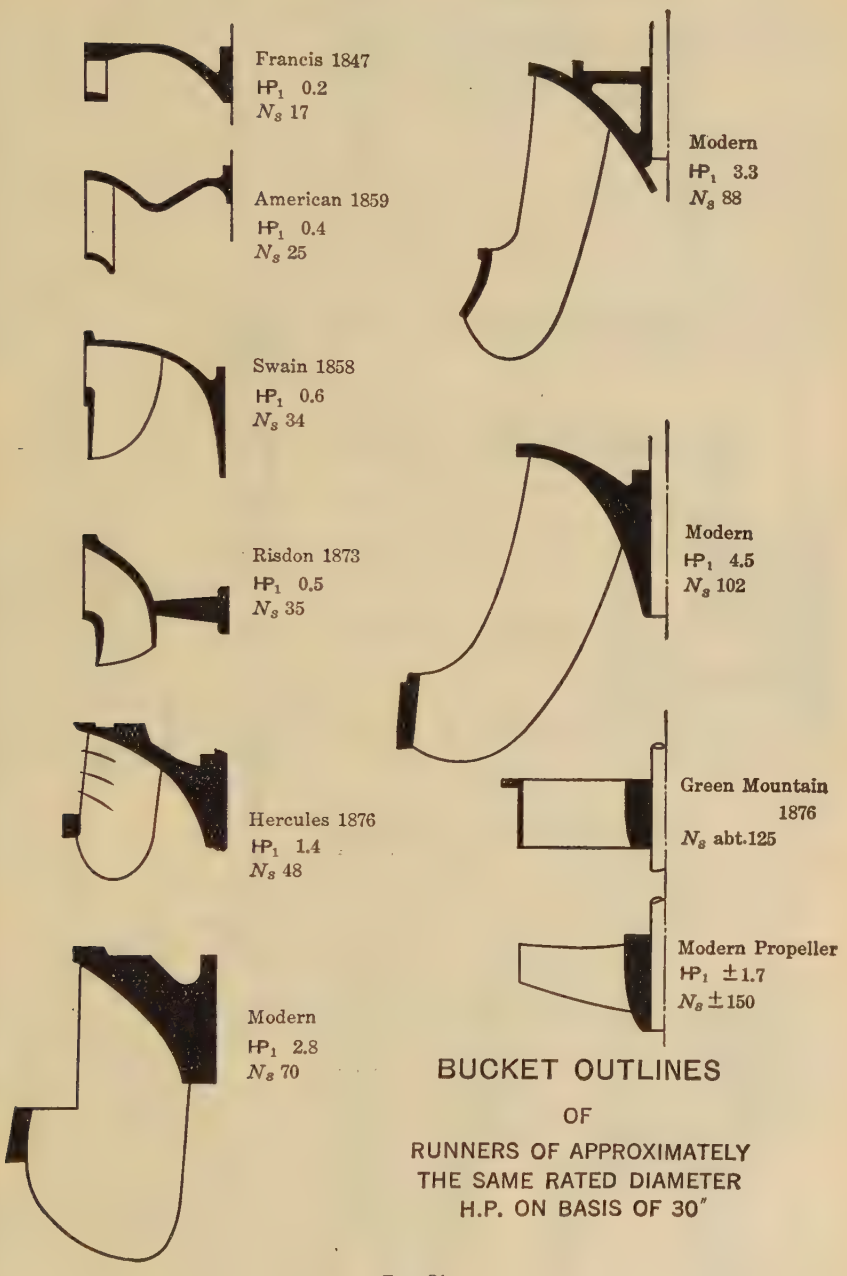

Francis 1847
HP1 0.2
Ns 17
American 1859
HP1 0.4
Ns 25
Swain 1858
HP1 0.6
Ns 34
Risdon 1873
HP1 0.5
Ns 35
Hercules 1876
HP1 1.4
Ns 48
Modern
HP1 2.8
Ns 70
Modern
HP1 3.3
Ns 88
Modern
HP1 4.5
Ns 102
Green Mountain
1876
Ns abt.125
Modern Propeller
HP1 ±1.7
Ns ±150
BUCKET OUTLINES
OF
RUNNERS OF APPROXIMATELY
THE SAME RATED DIAMETER
H.P. ON BASIS OF 30″

FIG. 21.

TABLE 1.—Characteristics of American Water-Wheels, 1847–1900, on Basis of 30-Inch Wheel.

Date.	Type.	1-Ft. Head.			Efficiency.			N_s (full power.)	Remarks.
		Revolutions per minute at maximum efficiency.	Horse-power, maximum.	Horse-power at maximum efficiency.	Full power.	Half power.	Maximum.		
1851	Boyden	38.3	0.47	....	79.4	46	79.4	26	Tremont turbine, inside diameter of 30 in.
1847	First Francis Model	44	0.19	....	....	....	71.6	19	2 13/16-in. bucket height; no gate.
1849	Boott-Francis	39	0.20	....	79.7	(55)	79.7	17	First Francis wheel installed.
1869	Swain	44	0.59	0.59	82.2	67.7	82.2	34	H. F. Mills, Lowell, 42-in. wheel, 5.35-in. buckets.
1870	Swain	45	0.69	0.69	81.9	....	81.9	37	do. Same model. Buckets. 7.4 in; lower band lowered; all else (except gate) the same.
1874	Swain	46	0.76	0.73	83.5	72.1	83.85	40	James B. Francis. Test on present Boott No. 2 on wheel shaft; no draft-tube.
1872	"American"	38.5	0.41	0.33	76.8	72	80.4	25	Emerson at Holyoke.
1871	Leffel	43	0.48	....	74.3	(60)	74.3	30	Emerson at Lowell.
1873	Tyler Scroll	44	0.42	....	81.6	57	81.6	29	" " Holyoke
1873	Houston	44	0.52	....	88	54	88	32	" " "
1873	Risdon	49	0.51	....	90.5	73	90.5	35	" " " , with diffuser.
1873	Risdon	50	0.49	....	88.8	....	88.8	35	" " " , without diffuser.
1876	"Hercules"	40	1.44	1.29	83.8	73	89.2	48	" " "
1878	"Victor"	42	1.32	....	86.7	65	86.7	48	" " "
1883	"Hercules."	40	1.44	1.28	85.14	....	86.94	48	In present Holyoke Flume.
1897	"Samson"	51	1.85	1.71	82.03	75.6	84.30	69	Holyoke Test No. 979
1897	Special "Hercules"	40	1.79	....	84.0	70.5	84.0	55	" " " 1051
1897	"Victor"	42	1.80	1.72	80.59	68.8	81.07	56	" " " 1061
1899	Smith-McCormick	53	1.65	....	84.55	69.3	84.55	68	" " " 1201
1876	"Green Mountain" Propeller	126	0.98	....	....	....	(60)	125	Calculated from maker's tables; 10-ft. head readings taken for 28 and 32-in. wheels used and averaged; water and speed assumed to be correct; wheel efficiency assumed at 60% and horse-power calculated from this.

The Scroll Setting

Many of the early turbines, such as the Boyden and the Swain, were built integral with their flumes. With the appearance of the small turbine, various settings were evolved. In the most common setting the wheel was placed directly in a hole in the floor of the flume which in some cases was widened at the end to form a box-like structure. By 1870, many of the small wheels were set in cast-iron globe-shaped cases and supplied by a pipe; this was particularly common with the higher heads. As the size of the wheel increased, the cost of cast-iron casings became, in most cases, prohibitive. Many of these early settings were excellent hydraulically, and often were quite as good as could be designed. In later years, however, as wheels increased in size, sheet-iron cases began to be used. All knowledge of hydraulics apparently was thrown to the winds, and structural conditions determined the shape of the casing. The boiler-maker ruled.

Many of the early wheels were set in wooden scroll cases (Fig. 22) and had no guides. Fig. 23 shows a type of wheel quite common for many years. It was used much in saw-mills as an auxiliary wheel for running back the log carriage. This diagram was copied from a drawing found among some of Boyden's papers, in the files of the Amoskeag Manufacturing Company, of Manchester, N. H. The construction of a similar wheel is described by Craik.*

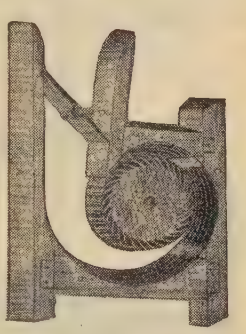


Fig. 22.—Rich Scroll, 1848.

After 1850, many scroll wheels came into use. They usually were of iron and were entirely self-contained, that is, the scroll was built integral with

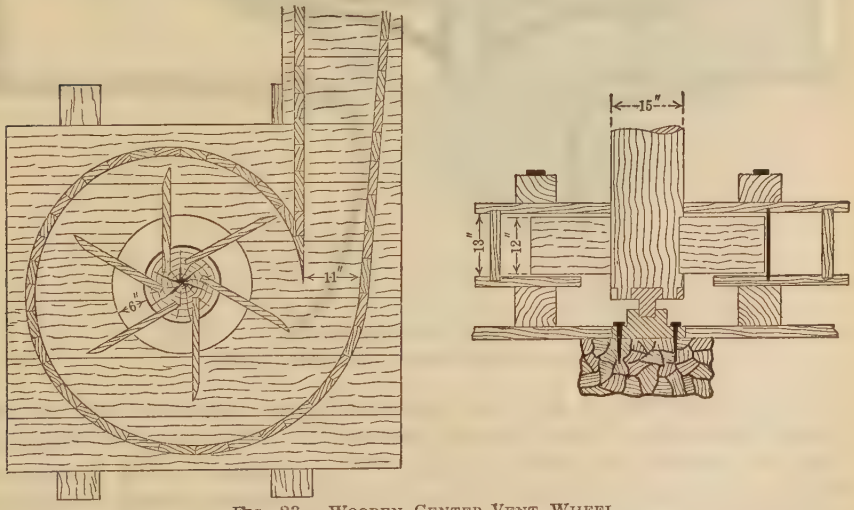


Fig. 23.—Wooden Center-Vent Wheel.

* "The Practical American Millwright and Miller", p. 205, Phila., 1870.

the bearings which supported the runner. (Fig. 24.) The gate was usually a plain sliding rabbit-trap, at the entrance to the scroll, although a butterfly was also used. These wheels had no guides, and were very poor at part gate, although at full gate they gave an efficiency of about 70 per cent.

Fig. 25 shows a model of a scroll wheel patented by Daniel T. Lakin, of Hancock, N. H., in October, 1863. Its runner was of the true Howd-Francis type, with flat blades, and was inverted so that it discharged upward. The interesting feature of this wheel was the regulating mechanism which was a register gate, fitted inside the runner and revolved with it. Closing the gate reduced the discharge area of the buckets and decreased the flow. Directly attached to the shaft are the flyballs which actuate the gate. The whole regulating mechanism is part of the runner and shaft, and the entire wheel is self-contained. There is no question but that such an arrangement would give poor part-gate efficiencies, due to contraction losses through the gate, but it is most interesting as an early example of the present practice of placing the flyballs directly on the shaft of a vertical unit.

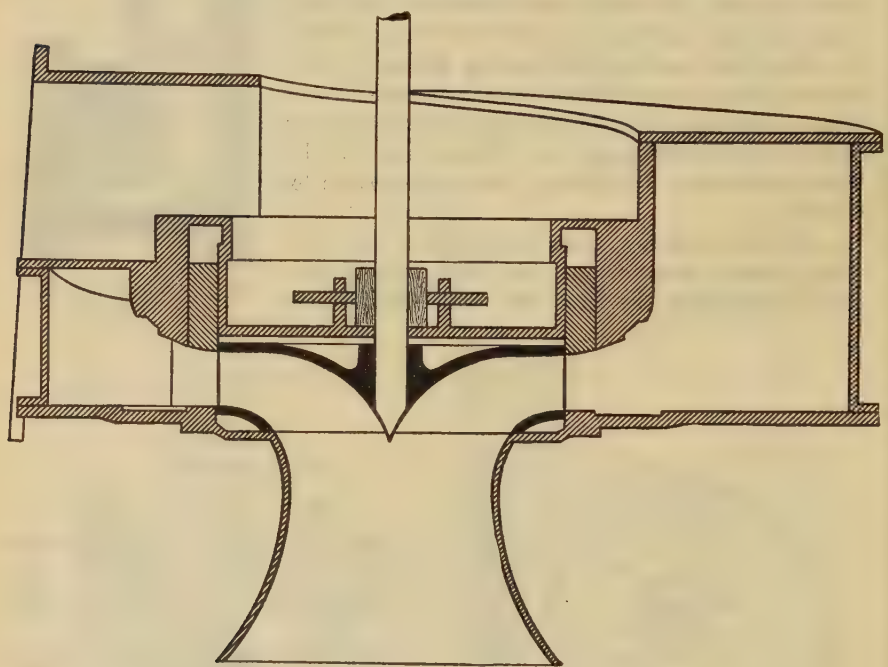


FIG. 24.—SCROLL WHEEL BY BOYDEN, 1854.

Probably the best of the scroll wheels, and the one which survived the longest, was made by John Tyler, of Lebanon, N. H., grandson of Benjamin Tyler who patented the "Wry Fly" in 1804. As made, in 1873, it gave an efficiency of 81.6% and a specific speed of 29. The same runner placed in a register-gate flume casing, gave a lower efficiency than when used in the scroll. A scroll was sometimes divided into two compartments by a thin horizontal

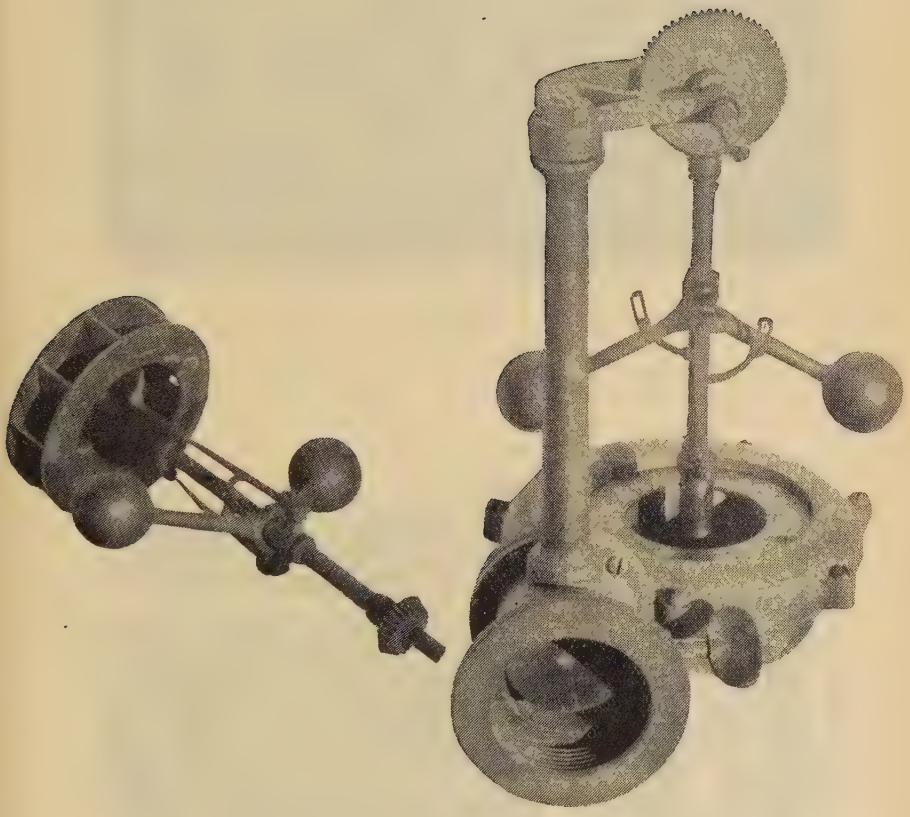


Fig. 25.—Lakin Wheel, 1863.

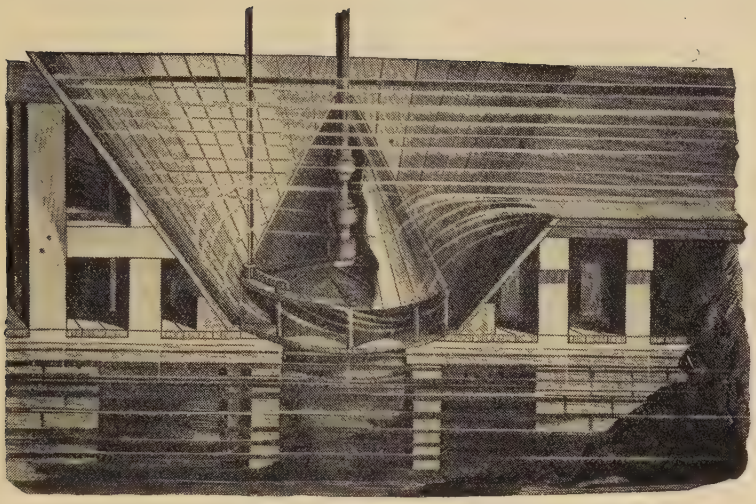


FIG. 26.—DROUILLARD FLUME, 1880.

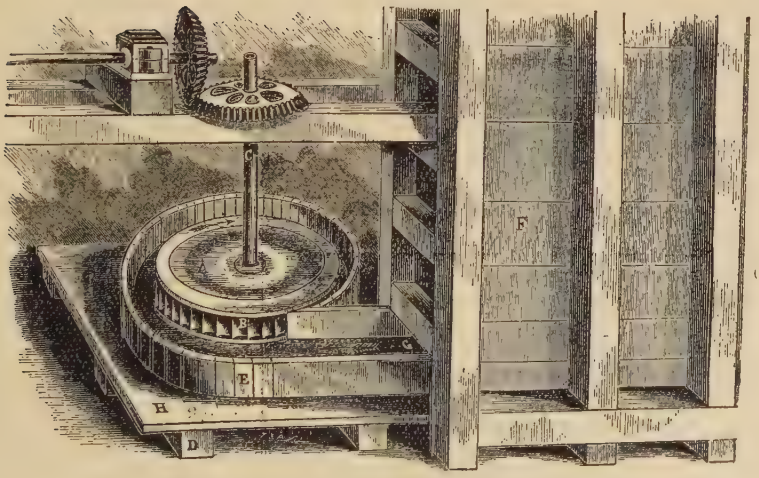
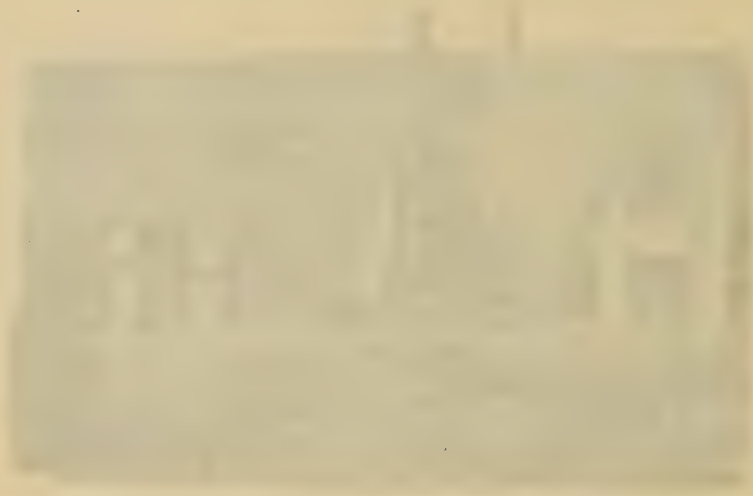


FIG. 27.—WARREN SCROLL, 1860.



diaphragm, so that, with the gate partly shut, the same velocity and direction of water might be retained in the lower half of the scroll. During the "cut and try" period, the iron scroll casing was used by many makers, especially with smaller wheels, but, by about 1880, they all had disappeared with the exception of a few, such as the Chase which, however, was not a true scroll wheel. The Drouillard flume (Fig. 26) was an attempt to improve the open-flume type of setting.

The time had now come in which, the increasing size of the wheels rendering the cast-iron case prohibitive in cost to most makers, and a setting more permanent than the wooden flume being desired, wheel casings of sheet iron came into use. About this time, the use of the draft-tube enabled the wheels to be placed above the tail-water, where they were more easily accessible for inspection and repair. For many years, almost every wheel maker considered the draft-tube merely as a device by which the wheel could be raised clear of the water and still retain the full head, and it appears that only one maker, Risdon and Company, appreciated its value in regaining velocity head. The development of the draft-tube will be discussed later.

With the coming of the boiler-maker period, the scroll setting disappeared, and it seems to have been almost entirely forgotten. Indeed one wheel maker of the present day is somewhat surprised that "examples have been found in what is left of some early mills in Connecticut of turbines of primitive construction, equipped with a volute form of water passage surrounding the runner."

Not only was the scroll used largely during the early years of the development of the American turbine, but its principle also was studied and understood. An article* by A. G. Hillberg, on scroll design, described a method of dividing the water used by the wheel, into a number of imaginary lines of flow, and of carrying these back through the scroll, thus studying the effects of shape. In the files of the Proprietors of Locks and Canals, there is a study, presumably by Francis, dated 1857, of a scroll design for the grist-mill wheel in which this same method is used.

The American Water Wheel Company, of Wareham, Mass., makers of the Warren wheel, were early users of the scroll case. (Fig. 27.) In a letter to the Committee of the Ninth Exhibition of the Massachusetts Charitable Mechanics Association, dated September 17th, 1860, that Company discusses its theory of scroll design. A few quotations are given, as follows:

"Our improved scroll is formed upon a plan which enables the water to pass, at the periphery of the wheel, at an uniform velocity at all parts of the wheel's circumference. The water in the scroll passes (or should pass) towards the center of the wheel with an accelerated velocity in proportion to the diminution of the area of the circles over which it passes."

The circles mentioned are a series of concentric circles, of area ratios of 1 : 2 : 3, etc., used in the construction of the scroll by graphic methods. This means that the water is given a uniform centripetal acceleration for each unit of angular distance traversed.

* *Engineering Record*, October 3d, 1916.

"The principle of the centripetal action of the water in the scroll is the same as that of the vortex, or whirl-pool, accelerating the velocity of the water in proportion to the diminution of the area as it approaches the center."

After being abandoned in the United States, the scroll was taken up by the Germans, and largely used for high-head Francis runners. Pfarr credits Voith with being the first to build an iron, spiral-cased, wicket-gate turbine (1894).* This type of wheel was much used in German and Swiss practice, and when high-head developments began in this country it was brought back from abroad. The first hydro-electric station at Niagara Falls was equipped with Fourneyron turbines made in the United States after designs by Faesch and Piccard, of Geneva, Switzerland. It is interesting to know, however, that the first turbines running under the full fall at Niagara were built by an American firm, James Leffel and Company, after the designs of A. F. Sparks.

In the earlier years of high-head development, the scroll was sometimes used, but a great many installations were of the boiler-maker type. By about 1904, however, the spiral casing was in general use for high-head wheels.

In 1903, a vertical unit with a spiral, steel-plate scroll was placed in a plant on the River Glommen at Kykkelsrud Falls in Norway, under a head of about 60 ft.† This appears to have been the first example, on a large scale, of what is now the best modern practice.

Until reinforced concrete made the modern setting possible, wood and cast or plate iron were all that could be used, aside from masonry. Metal cases were used, but, for a big unit, a plate scroll would have been considerable of an undertaking. It was much easier to build some kind of a plate casing, usually circular in form, around the wheel and to let it go at that. Efficiency was not sought as it is to-day, and in the days of direct drive and small units, a few per cent. did not mean very much. With the appearance of central stations and large units, even 1% meant money, and over-all efficiency was considered as it never had been before. By about 1900, and for a considerable time thereafter, a majority of the plants had horizontal shaft units, which either drove generators or were direct-connected to machinery. Electrification called for a fairly high speed, and this was accomplished by using batteries of runners of small diameter. Until the high-speed wheel was developed further, no other alternative was possible, as long as high speed was required.

The need for high speeds developed the runner and resulted in specific speeds of almost double those of the early wheels, such as the "Hercules" and "Victor". Because of this, it soon became possible, even with low heads, to drive large hydro-electric units by a single runner. Three elements—the development of the runner, the need for high efficiency, and the growth of the use of reinforced concrete—made possible the modern vertical unit, which appeared in this country about 1912. The first plans for Keokuk, Iowa, were for double-runner vertical units.‡ At present, the single vertical runner is used almost entirely for all low-head developments, and even high-head Francis runners have recently been placed in vertical cast-iron spiral settings, such

* "Turbinen für Wasserkraftbetrieb," Atlas, Plate XXVII, Berlin, 1907.
† Koester, "Hydroelectric Developments and Engineering", p. 382, N. Y., 1909.
‡ *Engineering News*, September 28th, 1911.

as the Big Creek No. 8, 680-ft. development of the Southern California Edison Company. Fig. 28 shows an installation of 58-in. wheels operating under a head of about 37 ft., at the Gardner's Falls plant of the Greenfield (Mass.) Electric Light and Power Company. This plant was built in 1913, and the acceptance test showed a wheel efficiency of more than 94 per cent.

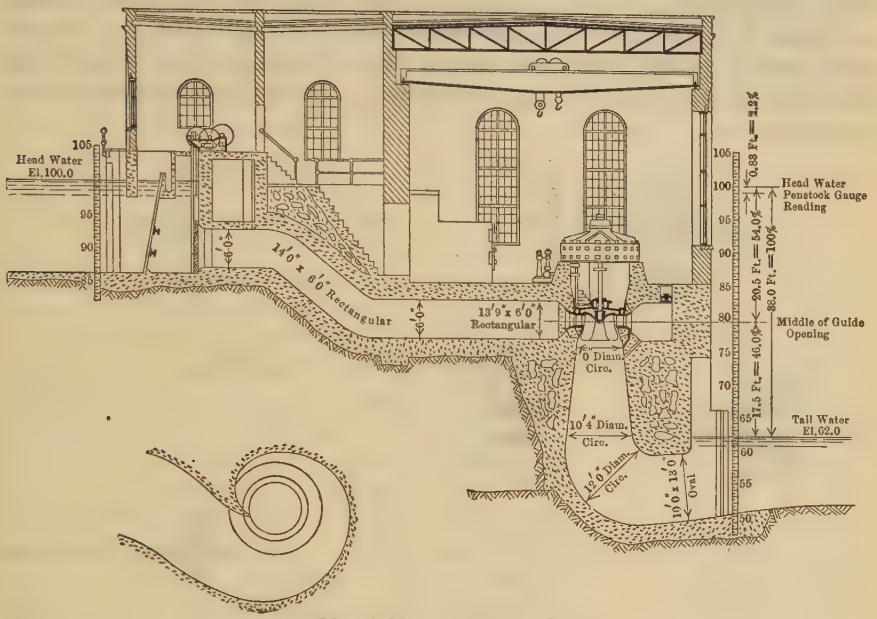


FIG. 28.—A MODERN WHEEL SETTING.

THE HORIZONTAL WHEEL AND ITS SETTING

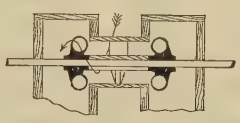
During the early days of the turbine, it was a natural step from the horizontal shaft of the breast-wheel to the placing of turbines in the same position. This setting was commonly used until about 1850. The wheels were usually of the outward discharge type, such as the spiral and Rose wheels, and often discharged into the open air. The double Parker wheel discharged outward into two wooden draft-boxes. (Fig. 29.) The efficient use of horizontal wheels required a draft-tube and until the draft-tube came into use this type of setting was not practical.

About 1880, the horizontal setting began again to appear. Gates Curtis seems to have been the first to use it. In 1879, a pair of Curtis horizontal wheels were installed in the Hudson River Mill at Palmer's Falls, N. Y. They were placed directly on the horizontal machine shaft, 16 ft. above the tail-race. A square wooden draft-tube was used and the head-water stood 12 ft. above the center line of the wheels. In a letter to the wheel maker, Mr. Curtis states:

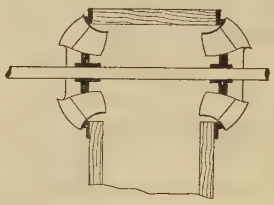
"We are satisfied by experiments made with pipes leading from the top and other parts of the draft-tube attached to glass and mercury gauges that we get equal results from the wheels, as we would were they placed at the

level of the tail-water. This arrangement of taking off power so far above the tail-water, thereby avoiding the annoyance and expense of bevel gearing, is proving most satisfactory in many ways * * *."

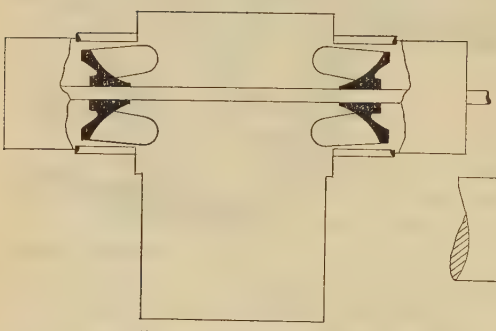
A pair of horizontal Curtis wheels (Fig. 29) were tested at Holyoke in 1879, and found to give an efficiency of 72%, whereas, on a vertical shaft, one alone gave 83.6 per cent. About 1882, the Humphrey Machine Company, of Keene, N. H., began to build horizontal double units in cast-iron "camel-back" cases of rather fair design, and, by 1890, this type of unit, constructed of boiler-plate, was in general use. Hydraulically, the majority of these settings were very poor.



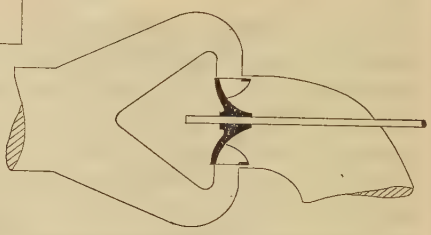
Parker 1828



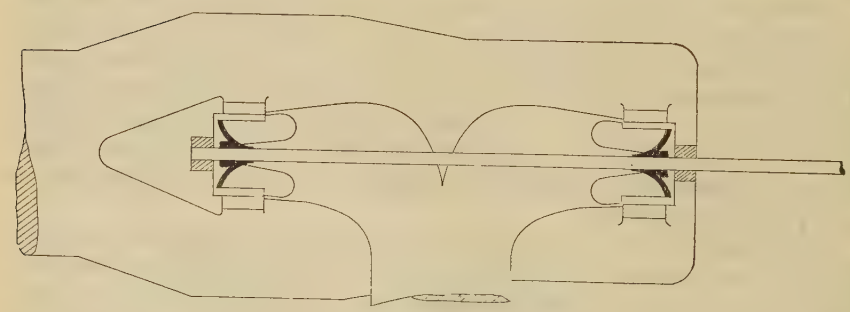
Curtis 1879



"Boiler maker" Setting 1890



Cone Setting



Modern Horizontal Setting

FIG. 29.

Some double horizontal settings were made of cast iron, such as the Risdon wheels in the Jefferson Mills of the Amoskeag Company of Manchester, N. H., J. B. Francis, Consulting Engineer, but they were much the exception to the rule. These wheels were interesting, because on one shaft were runners operating under two different heads. There were other good horizontal settings,

such as that of the Lawrence Company of Lowell, placed in 1895 (Fig. 30), but they were not common.

In many of the horizontal units built in later years, the casing was a continuation of the penstock, and the water approached the wheels parallel to the shaft. Such a setting, with a correctly designed casing, was a great improvement over the earlier ones in which the water entered a cramped casing with sharp corners, at right angles to the wheel shaft. Fortunately, however, increase in runner speeds and the use of reinforced concrete permitted the beginning of a new period in wheel settings and the rule of the boiler-maker is now a thing of the past.

Probably the worst examples of horizontal wheels appeared during the Nineties. (Fig. 29.) The setting usually consisted of either a rectangular box or a tube. The wheels were placed at each end, and discharged toward each other. A round metal draft-tube extended from the bottom of the draft-box into the tail-water. Every opportunity for eddies and whirls was offered by the draft-box and tube, and sharp edges and corners abounded everywhere. The worst part of this type of setting was that the water was not conducted smoothly to the draft-tube. It was discharged at a fairly high velocity into a large box where it swirled around and was allowed to find its way out through the hole in the bottom of the casing. Sometimes each wheel had its own quarter turn and draft-tube and was set in a common casing, but, usually, both discharged into the same tube. One wheel catalogue shows an ordinary double horizontal setting, to which had been added a third unit with a vertical shaft. This unit discharged downward and was placed directly above and concentric with the draft-tube opening.

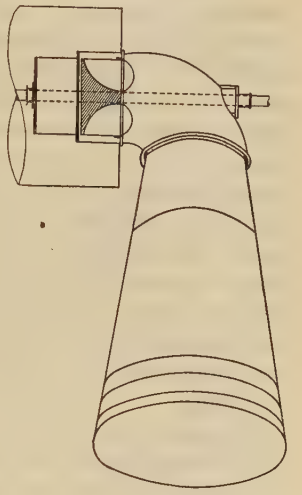


FIG. 30.

Perhaps the worst feature of the earlier horizontal wheels was the proximity of the runners, when both discharged inward against each other. In some cases, the distance from the bottom of one runner to the other was as small as one wheel diameter. In the early days of the boiler-maker period, this distance was usually from 1.5 to 2 runner diameters. In 1894, at the plant of the Appleton Company of Lowell, a test on Rodney Hunt wheels showed that insufficient distance between runners reduced the efficiency 10 per cent. A pair of 30-in. "Hercules" wheels were tested for the Middlesex Company of Lowell in 1896. As tried at first, the runners were 2.26 diameters apart. By increasing this distance, 16 in., or to 2.8 diameters, the maximum efficiency was increased 4.7 per cent. Modern runners, operating under medium to low heads, should be spaced from 3 to 4 diameters apart.

The "camel-back" type of cast-iron draft-chest was gradually evolved, and showed a great hydraulic improvement. The discharges of the two wheels, instead of being directly toward each other into an open case, were gradually

turned downward by a curving partition wall so that they were flowing in a more or less parallel direction when they met. Swain devised the idea of carrying this dividing partition all the way down, thus, in effect, giving each wheel a separate draft-tube. Fig. 29 shows the cone setting for a single horizontal wheel. This excellent design for small units under fairly high heads was used by several makers.

The double horizontal wheel in a cast-iron, "camel-back" draft-chest, when placed in a casing of good design and fitted with a properly flaring draft-tube, was an excellent prime mover. The best practice usually was to place this wheel in a casing that was a continuation of the penstock, so that the flow was parallel to the wheel-shaft. The approach to the wheel was eased by placing a cone-shaped casing over the up-stream end of the unit. In a setting of this kind, the water passage around the first wheel was sometimes cramped, and the flow to the down-stream wheel was restricted. To obviate any possibility of this, the casing was sometimes enlarged at the point opposite the gates of the up-stream wheel. (Fig. 29.) A horizontal setting of Smith wheels in a casing of this kind was placed at the plant of the International Paper Company, at Glens Falls, N. Y. These wheels were tested in 1918 and showed an efficiency of 88%, which is thought to be the highest authentic result ever obtained from a double horizontal setting. The best results that can be obtained from such a unit will always be from 2 to 4% less than the efficiency obtained from one of the runners tested, on a vertical shaft, under the best conditions. There are some places, however, where efficiency is not the primary requisite, and certain advantages of the horizontal setting make it suitable.

The horizontal setting, to a very large extent, is vanishing, but there are still certain cases to which it is pre-eminently fitted. The most important of these is found in pulp mills, where the wheels are direct-connected to the horizontal shaft of a battery of grinders. Conditions call for a fairly high speed, regardless of the available head and the multi-runner unit is often the best adapted to the need.

The Draft-Tube

In June, 1840, Zebulon and Austin Parker, of Licking County, Ohio, were granted a patent for a "Draft-Tube for Water-Wheels". Parker wheels, usually, if not always, were set in pairs on a horizontal shaft, fed by a crude scroll case, and each discharged outward into its own draft-tube which was merely a long rectangular wooden box. The object of this setting was not to gain efficiency, but to allow the wheels to be placed above the tail-water. The French Jonval-Koechlin turbine, in 1841, was equipped with a draft-tube, but the Parkers seem to have been the first to apply it.

The first example of the use of a draft-tube to regain velocity head was the "diffuser" of Boyden (Fig. 31), which, by 1846 was being applied to most of the Boyden wheels built. It was shown to add about 3% to the wheel efficiency, by regaining the velocity head of the discharged water. Theoretically, it should have saved almost 5%, so the diffuser efficiency was about 60 per

cent. The flare of the parallel disks was apparently too sudden, and the water failed to cling to the sides of the diffuser, with the result that the discharge was a succession of surges. In later years, Boyden wheels were not equipped with diffusers, probably because their great size required unnecessarily large wheel-pits which, in many cases, might have required the changing of the foundations of the mill. Only one diffuser is left in Lowell at the present time; it is on the 1853 Boyden wheel at the plant of the Merrimack Manufacturing Company.

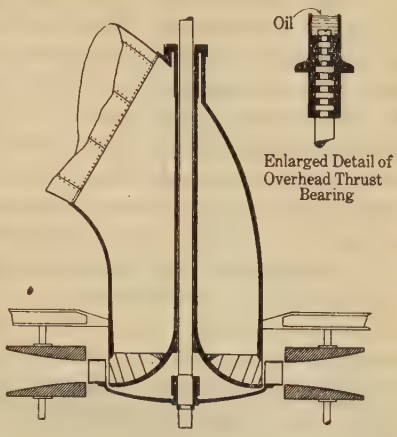


FIG. 31.—BOYDEN WHEEL AND DIFFUSER.

The draft-tube, as a diffuser, does not appear again for many years. By 1870, many wheels were set above tail-water on wooden stave or sheet metal tubes. Sometimes, the construction of the hooped wooden tubes required a slight flare, but usually they were straight. The design, if there was any, was usually based on the velocity with which a bubble of air would rise in water. In 1881, Gates Curtis, maker of the Curtis wheel, recommended the proportioning of draft-tubes for a velocity of from 5 to 8 ft. per sec. The general feeling among wheel makers was that a wheel could be used with a draft-tube with little or no loss in efficiency, provided the tube was air-tight.

About 1880, James Emerson made a series of experiments on draft-tubes, using a 15-in. Victor wheel. He began with a draft-tube the inside diameter of which was the same as the wheel skirt, and then, by inserting fillers, reduced the area by degrees. The velocities in the tube varied from about 5.5 to 12 ft. per sec., with the best efficiency at the lower velocity, and this efficiency was considerably less than that obtained when the wheel was tested without a draft-tube. The draft-tubes used were all straight sided, and were from about 7 to 10 ft. long.

At Holyoke, in 1873, the Risdon Company tested a 36-in. wheel which gave an efficiency of 90.5% with a draft-tube diffuser and 88.8% without it. This trial must have convinced the Risdon Company of the efficacy of the appliance, for it built draft-tubes of excellent design during the period when other makers were evolving weird affairs of boiler-plate.

The Risdon runner discharged axially over an annular section, and there was a dead area around the center of the wheel. To eliminate this, an inverted cone was fitted below the runner, so that the discharge passage was smoothed. The draft-tube, which was slightly flared and usually of cast iron, was excellent and far ahead of its contemporaries. It was claimed that the efficiency of the wheel was increased 2% by the use of the diffuser. However, during the period of the double horizontal unit, even the Risdon draft-tubes fell down, and examples are found of two wheels discharging against each other in a cramped wooden draft-chest.

In the days of direct drive, a difference of 2 or 3% in wheel efficiency did not mean a great deal. The wheels were of such low specific speeds, that the total gain realized by the use of a draft-tube was only 2 or 3%, and the gain, in many cases, did not justify the expense of building a good draft-tube. It is interesting to note that "W. W. T.", writing of the "Green Mountain" propeller wheel, realized that, with a wheel of this type, a draft-tube would be of greatly increased value. He says: "If Mr. Boyden would put his diffuser on one of these wheels, we would promise him a gain of more than 3 per cent. It would be more nearly 50".*

It would appear that during the period from 1870 to about 1900, or a little later, the use of the draft-tube, as a diffuser, was generally understood, but it was not generally used. The dollars and cents value of a draft-tube usually did not justify the necessary outlay.

With the coming of electrification and higher specific speeds, the draft-tube became of greater potential value, and the use of reinforced concrete made it a simple matter to build a tube of any desired shape. The development of large central power stations meant that even 1%, when translated into kilowatt-hours per year, amounted to a considerable sum. The draft-tube was more than justified economically, and its use, as a diffuser, became universal.

Apparently, the pendulum has swung to the other extreme, and there are now on the market various kinds of patent draft-tubes, each of which, like the old patent medicines, is promised to be the cure-all for every hydraulic trouble. The basis of one of these types is an inverted cone placed beneath the discharge opening of the draft-tube, with its axis coincident with that of the runner. The object is to change smoothly and without shock the direction of the water from a vertical to a horizontal direction.

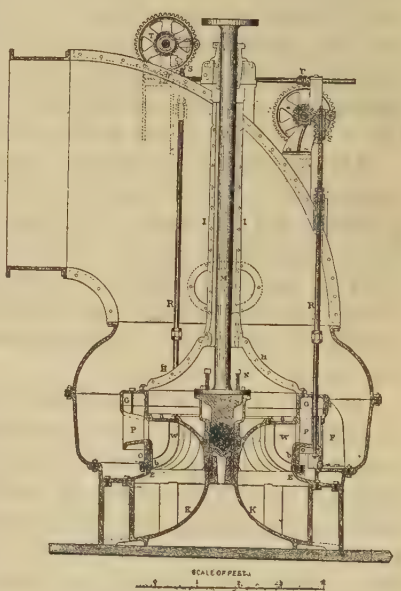


FIG. 32.—SWAIN WHEEL, 1869.

The placing of a cone beneath the draft-tube is almost as old as the turbine itself. Some of the early Jonval-Koechlin turbines had it. Almost every German textbook on water-wheels from 1860 to Dr. Camerer's "Wasserkraftmaschinen", of 1914, shows one or more settings with cones below the draft-tube. English and French textbooks have the same type of setting, and Beardsley shows one.† The Swain wheel had a lower step bearing, and, by 1869, this had been smoothed off to form a cone (Fig. 32). Fig. 33 shows a Swain wheel, placed in 1875, at the Boott Cotton Mills, Lowell, Mass. To

* *Emerson's Turbine Reporter*, January, 1876.

† "Hydro-Electric Plants", p. 344. N. Y., 1907.

FIG. 33.—SWAIN WHEEL AT BOOTT COTTON MILLS.

better conditions, the cone was continued, as shown, by wooden planks, and, in another instance, at the plant of the Hamilton Manufacturing Company of the same city, it is made of concrete. The iron-cased Burnham wheels of about 1890, had a metal cone below the runner.

Fig. 34 shows a wheel setting erected by the T. H. Risdon Company at the Lawrence Woolen Company, Lawrence, Mass., in 1883. In the maker's catalogue (undated), of about this period, it is stated:

"This wheel and the two preceding it have diffusers. In the Lawrence Woolen Company, a very shallow pit was already constructed, in which the water only stood about 18 in. deep. If the end of the draft-tube had been set low and had discharged the water vertically, as in the preceding wheels, it could not have escaped freely, and there would have been a corresponding loss of head. Hence, the upper part of the tube was constructed as in the previous case, but the lower end has an outward discharge. We have carefully constructed diffusers, both with downward and outward discharges, and to obtain clear evidence of what the gain was, have tried the same wheel, both with and without a diffuser, and in every case have found a gain.

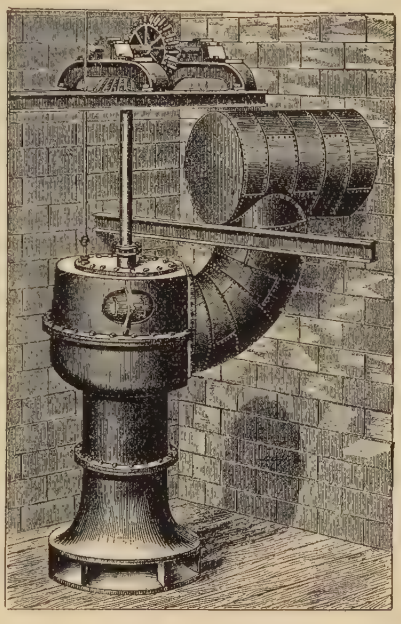


Fig. 34.—Risdon Diffuser.

"The increased economy of water obtained by a diffuser is 2 per cent. When the water leaves the water-wheel, it is traveling with a rapid velocity of about 3% of that due the whole fall. Of this loss, 2% may be saved by a diffuser. If the wheel has 200 h. p., 4 h. p. would be saved, and, if it must be made up by steam, there would be a saving of about $200 the first year, which would more than pay for the diffuser.

"Hence, where great economy of water is desired, a diffuser is a good investment. Every year we make more of them than in previous years. This arrangement here shown is the best possible for many locations, and, under such circumstances, we prefer a diffuser to any other arrangement. We always build the diffuser of cast iron, believing that the only way to obtain a satisfactory arrangement."

Water-Wheel Testing

The question has arisen, at various times, as to the accuracy of results obtained at the present Holyoke Testing Flume. Before discussing this matter, a rough survey will be made of all the large-scale turbine tests of record. Such tests as those made by Francis, on the Tremont turbine (Boyden), the Boott center-vent, the Boott Swain, and the Tremont and Suffolk Humphrey are above suspicion. They were carried out under scientific conditions, with practically no regard for expense, by the most eminent hydraulic engineer in the United States, if not in the world. The test of a 42-in. Swain wheel,

at Lowell, by Mills, in 1869, has been questioned by James Emerson, but unjustly it would appear, and without good reason.

In 1859-60, the City of Philadelphia conducted a series of tests at the Fairmount Water-Works. They are understood to have been rather crude, and the results are not considered trustworthy. Instead of using a Prony brake for the measurement of power, a method was used whereby the wheel hoisted a given weight. The wheels tested were Jonvals and early scroll wheels; a Jonval built by Stevenson gave the best results.

The wheel tests at the Centennial Exhibition in 1876 were tested under the direction of Samuel Webber. The highest results were given by the Risdon wheel. The following is quoted from Mr. Webber's discussion of the paper by the late R. H. Thurston, M. Am. Soc. C. E., on "The Systematic Testing of Turbine Water-Wheels in the United States", before the American Society of Mechanical Engineers:*

"There have been some remarkable tests reported from the Old Flume [Emerson's], which it has been impossible to repeat or duplicate at a later date with the same wheels, and the 90% test of the Risdon wheel, referred to by Professor Thurston, is one of them. I have no doubt of the correctness of the Centennial test, which gave 87.68% net effect from this wheel, for other wheels tested by other engineers, in various places, corroborate it very closely, but I have never myself got so high a result from any other wheel; and it should also be noted that the very high efficiency reported from some wheels has been usually found in the tests of very small wheels, of 15 or 20-in. diameter, where a very considerable effect might be exercised upon the wheel by the man who handled the lever at the brake."

In the same discussion, another view is given as to the accuracy of the 1876 tests, as stated by the late Charles E. Emery, M. Am. Soc. C. E. After describing how petty politics resulted in the appointment of Mr. Webber, he says:

"As the only object was to obtain reliable and creditable results, the writer undertook to co-operate with Mr. Webber in the conduct of the work. This, however, did not prove an easy task, as Mr. Webber had made a great many tests, had acquired certain methods of his own, and did not care to go into many of the refinements which such an opportunity would have made of great scientific value. While he was pleased with the action of the Judges, he evidently felt, moreover, that his authority was from another source, and insisted practically on having his own way, and under the circumstances little else could be done. The original design was such that the water was necessarily admitted to the weir approach at the side, so that, although ample provision had been made in the length of the approach, the current on the side opposite the inlet was very much the stronger. The speaker suggested a series of baffling screens or racks, such as are described in Mr. Francis' work, and finally a single one was hastily applied, but with the spaces between the bars so wide that there was still ample area for the water with greater velocity to pass along the side where it did before. Any further improvements were, however, considered unnecessary by Mr. Webber, and the experiments were conducted with the water approaching the weir at very different velocities on the two sides; and, moreover, on account of the recoil of the current from

* *Transactions*, Am. Soc. Mech. Engrs., Vol. VIII (1887), p. 359.

one side of the approach, floating bits of wood showed that all of the water did not approach the weir at a right angle, and the evidence of this deflected current was sufficiently marked to show a ridge in the crest of the fall at the weir itself."

James Emerson was a most interesting and outstanding character during the closing years of the "cut and try" period. He had been a sailor at one time, but soon after the close of the Civil War, he became interested in the testing of water-wheels. He was associated with early wheel tests at Lowell in 1869-71, and afterward moved to Holyoke, where he opened a commercial flume for the testing of wheels. The business prospered and many wheels were tested during the Seventies. A number of years were required to bring all the makers to a belief in the flume test, but Emerson's efforts were finally successful, as is shown by the prestige of the Holyoke test at the present day. James Emerson deserves the credit for the early development of the testing system, which was as necessary to the development of the reaction wheel as the "cut and try" period. One without the other would have been useless, but the combination of the two was what produced the "Hercules", the "Victor", the "McCormick", and the "Samson" turbines.

Emerson was a man of decided ideas and strong prejudices. His attitude, unfortunately, was that, if a maker did not have sufficient faith in his wheel to have it tested at Holyoke, which incidentally meant money in Emerson's pocket, the wheel was beneath his notice, and, as far as he was concerned, did not exist. Most of the wheels of the period, however, did go to his flume, and thanks to him, there is a rather complete list of tests on early American wheels. The absolute accuracy of his tests might well be questioned, but, comparatively, they must have been correct within 2 or 3 per cent.

Emerson was of extreme iconoclastic tendencies, and no institution in existence seemed to have escaped his abuse at some time or other. His most unusual book, "Hydraulics, Dynamics, etc",* treats of everything from technical matter concerning water-wheels to notes on the Bible, theology, law, woman suffrage, and spiritualism. He did much to educate the public by his annual reports and by his monthly magazine, *The Turbine Reporter*. Whatever were his bad points, his name is closely linked with the history of the development of the American mixed-flow turbine.

Emerson's testing flume was taken over by the Holyoke Water Power Company and, a little later, in 1881, the present testing flume was built under the direction of Clemens Herschel. The standards of the Holyoke test, from this time on, were the same as they are to-day. Concerning the accuracy of Emerson's tests, Mr. Herschel states:

"So far as I know, the Emerson tests are reliable, with the exception of some that must needs have failed, due to the routine, or mechanical, that is, hasty and perfunctory, manner in which they were in the course of time all conducted".

As the arrangement of the present Holyoke flume is well known and has been described by many writers, it will not be necessary to describe it in detail. The water is measured by a sharp crested weir with which little fault

* Willimansett, Mass., 1894.

can be found. The power is measured with a dynamometer of the Emerson type, consisting of a broad-faced cast-iron pulley, with a wood-lined metallic brake band, cooled by a constant flow of water. Several sizes of dynamometers are used, depending on the size of the wheel. With a heavy load, the brake seems likely to stick and work unevenly at the lower speeds. It is thought that a considerable improvement might result from the use of an Alden absorption dynamometer. The late Professor Thurston stated* that the limit of error at Emerson's flume was from 3 to 5%, and that:

"A careful examination * * * has led the writer to the conclusion that the Holyoke Testing Flume, and the methods of observation and calculation employed there, are capable of giving the efficiencies of turbines tested correctly within a limit of error of certainly less than one per cent, and probably to within one-half of one per cent. For all practical purposes, the results of the trials of turbine water-wheels, at the Holyoke flume, may be taken as exact, and absolutely trustworthy."

In conclusion, it is only fair to state that, although all available sources that gave promise of any useful information have been consulted, there may be a few omissions. An historical study is no better than the sources consulted, and, in the present case, the sources were, for the most part, scant and scattered.

Many thanks are due to Mr. S. S. Kent, Assistant Engineer, The Proprietors of the Locks and Canals, for his help in preparing this paper.

* "Systematic Testing of Water-Wheels", *Transactions*, Am. Soc. Mech. Engrs., Vol. VIII, p. 49.

DISCUSSION

B. F. Groat,* M. Am. Soc. C. E. (by letter).—There is no better means of becoming familiar with the fundamentals of any type of engineering development than by way of history. Development history is bound to be a chronicle of successes and failures, showing indisputably the effects of right and wrong efforts. The history of the turbine and its setting embraces the true science of its design. A little study of that science will convince any one that it is not essentially the result of individual effort, but the inevitable result of civilized effort under the influence of economic stress.

These truths are made clear by the authors, who essay to demonstrate the truth of the old adage, "there is nothing new under the sun," by showing that there is nothing new under the water. However, their paper discloses a record of advance that surely means positive improvement in things, which, although they may not be new, have been made to yield a new result, or record. Let credit then be given to the man who made the new record for that advance, whether it be large or small. There is nothing so discouraging to the scientific laborer as the failure of those about him to grasp his point of view. It may be vital to an intelligent appreciation of the value of his work. It is the cheapest remuneration he can be accorded.

The authors are certainly correct in their view that progress has been made by a multitude of small steps rather than by large strides. They also exhibit, in a striking manner, that some of the steps have been backward, only to be retraced contrariwise. In particular cases, it is sometimes hard to say whether the success or the failure has led to the positive advance. Thousands of supposed inventions are merely the results of ignorance, and may easily be shown to be only the discarded products of richer experiences of the past.

There is always the chance, however, that changed conditions may create a new demand for an old device to a new use. The draft-tube, as described by the authors, is a good example. At first, it was used apparently for the sole purpose of elevating the turbine above the tail-water. When scientific views broadened and economic conditions changed, the diffuser principle became better known, with the result that draft-tubes are now in general use.

It is doubtful whether there has been any great increase in efficiency, as applied to turbine runners *per se,* since the time of Boyden and Francis. Hydrodynamically, the efficiency of a turbine depends almost entirely on the entrance and exit angles of the buckets, supposing that water passages are smooth and curves gentle. Nothing proves this truth of science more certainly than the astonishingly uniform results of tests on good designs of the greatest variety. The curvature of the bucket is of little consequence as long as it leads easily from entrance to exit. This is one of those fundamental conditions of Nature which makes successful designers of turbines, just as a rich soil makes good farmers. No disrespect is intended for engineer or farmer; both must be proficient to secure the best results. The idea is merely to remind ourselves of the opportunity that Nature offers, namely, 100 per cent.

* Civ. and Hydr. Engr., Pittsburgh, Pa.

After the main features of turbine runners had been developed by the eminent engineers mentioned by the authors, and proper designs had been established, the succeeding considerable advances were to be looked for in the setting. Every form of runner requires its individual setting. It is in the setting that the greatest advances have been made during recent years. It is not improbable that any good runner of the older time, such as the Swain runner, shown on Fig. 12, with slight modification and a specially designed setting, could be made to yield an efficiency about as high as that of the best modern runners in their settings.

As to the classification of water-wheels, the list and method given in Table 2 were worked out by the writer at the University of Minnesota between 1901 and the time of Basshuus' classification.

Let N, P, and h, be the required speed, power, and given head for a water power development. Solve for $I = N^2 P \div h^{\frac{5}{2}}$. Divide I by the index of any wheel and the quotient will be the necessary number of such wheels. Multiply the speed coefficient by $h^{\frac{1}{2}} \div N$, and the result will be the required diameter of such wheels. Divide the total required power by the previously obtained number of wheels and the result will be the power of each wheel. In Table 2, the columns headed "Range" show the probable errors occasioned by using the corresponding coefficients, the various sizes of wheels of any given design varying more or less as to relative performance among themselves according to the design; the foregoing data have been taken from the published catalogs of the various makers and probably refer to performance at best speeds in all cases.

The data for any individual wheel given in Table 2 were calculated by students in the School of Mines. The "index" was determined by the equation:

$$i = \frac{N^2 P}{h^{\frac{5}{2}}}$$

in which N is the speed of the wheel, in revolutions per minute, P, the number of horse-powers, and h, the head, in feet, acting on the wheel. It is apparent that this is the square of the specific speed as determined by Basshuus.

Although the specific speed is a useful constant for the water-wheel designer, its square is equally useful to the hydraulic engineer who wishes to examine a number of types of wheel with reference to their applicability to a given water-power project. The use of this constant is explained in a preceding paragraph. D. W. Mead, M. Am. Soc. C. E., aptly calls it the "specific power" coefficient.* It is the power of a wheel of the same type that will run at unit speed under unit head.

It is surprising to know that the beveled buckets of the Houston wheel, showing an efficiency of 88%, have almost disappeared. It would seem that some such plan as this for receiving the water on the buckets would be appreciably better than the present plan, which involves a right-angle turn, although not so abrupt as a superficial view of an axial cross-section might suggest. Probably questions of construction have led to the abandonment

* "Water Power Engineering.

TABLE 2.—Partial List of American Water-Wheels with Average Coefficients for Each Type.

Reference.	Index.	Range, percentage.	Speed coefficient.	Range, percentage.	Number of wheels.	Name of wheel.	Manufacturer.	Location.
24	162	11	76.5	0.1	23	Victor High Pressure	Platt Iron Wks. Co.	Dayton, Ohio.
29	715	10	100	0.1	10	Water-Tight	Wm. Bartley & Sons	Bartley, N. J.
19	764	7	101.6	0.4	12	Syracuse	Alex. Bradley & Dun	Syracuse, N. Y.
17	850	13	102	0	22	High-Head Type	Dayton Globe Iron Wks. Co.	Dayton, Ohio.
40	910	..	102.3	...	1	Snoqualmie Type	Platt Iron Wks. Co.	Dayton, Ohio.
11	950	8	83.8	0.6	11	Little Giant Std. Flume	Munson Bros. Co.	Utica, N. Y.
3	1 020	20	112	0.7	3	Std. Cyl. Gate	S. Morgan Smith Co.	York, Pa.
21	1 030	1	111.5	0.05	7	Standard	James Leffel & Co.	Springfield, Ohio.
9	1 080	13	104.6	0.5	..	High Duty Std.	Risdon-Alcott Co.	Mt. Holly, N. J.
27	1 090	18	113.9	1	6	Special High Head	Trump Mfg. Co.	Springfield, Ohio.
10	1 120	13	84.2	0.4	..	Little Giant Std.	Munson Bros. Co.	Utica, N. Y.
13	1 120	8	83.9	0.6	6	Deep Flume	Munson Bros. Co.	Utica, N. Y.
36	1 190	15	110.7	2	21	National	Case Wheel & Mill Co.	Bristol, Conn.
14	1 200	17	115	0.1	20	Poole-Leffel	Poole Eng. & Mach. Co.	Baltimore, Md.
30	1 250	5	100	0.1	12	Special Water-Tight	Wm. Bartley & Sons	Bartley, N. J.
12	1 370	7	84	0.2	8	Deep Scroll	Munson Bros. Co.	Utica, N. Y.
4	1 460	9	112.9	2.2	..	L. C. Cyl. Gate	Risdon-Alcott Co.	Mt. Holly, N. J.
35	1 460	17	108.8	2.5	14	Bradway Patent	C. P. Bradway Mc. Wks	Camden, N. Y.
33	1 480	7	126.5	0.3	17	Flenniken	Dubuque Tur. & Roller Mill Co.	Dubuque, Iowa.
41	2 140	6	105.3	2	24	New Pattern Hunt	Rodney Hunt Ma. Co.	Orange, Mass.
5	2 220	5	112.4	3	..	D. C. Cyl. and Reg. Gate	Risdon-Alcott Tur. Co.	Mt. Holly, N. J.
8	2 250	13	104.4	1	..	Special High Duty	Risdon-Alcott Tur. Co.	Mt. Holly, N. J.
25	2 342	11	101.1	0.1	..	Victor Reg. Gate	Platt Iron Wks. Co.	Dayton, Ohio.
31	2 440	10	.....	...	6	"1900" Water Tight	Wm. Bartley & Sons	Bartley, N. J.
26	2 570	4	105.3	2	11	Crocker	E. D. Jones & Sons	Pittsfield, Mass.
18	2 600	7	102.5	3	19	McCormick's Holyoke	Rodney Hunt Mch. Co.	Orange, Mass.
32	2 660	8	102.7	3	19	" " "	Dubuque Tur. & Roller Mill Co.	Dubuque, Iowa.
2	2 720	7	102.7	3.2	..	McCormick	S. Morgan Smith Co.	York, Pa.
15	2 910	3	107	0	18	New Am. Special	Dayton Globe Iron Wks. Co.	Dayton, Ohio.
20	3 430	3	132.9	2	7	Samson (1st set)	James Leffel & Co.	Springfield, Mass.
34	3 990	2½	117.2	0.6	16	United States	Camden Wt. Wh. Wks.	Camden, N. Y.
22	4 020	1¼	116.6	0.8	17	Victor Std. Cyl. Gate	Platt Iron Wks. Co.	Dayton, Ohio.
28	4 110	¾	112.2	0.1	14	Standard	Trump Mfg. Co.	Springfield, Ohio.
23	4 440	¾	117	0.5	17	Victor Inc., Cap. Cyl. Gate	Platt Iron Wks. Co.	Dayton, Ohio.
20	5 360	¼	135.6	2	10	Samson (2d set)	James Leffel & Co.	Springfield, Ohio.
7	5 530	½	143	0.2	..	Leviathan	Risdon-Alcott Tur. Co.	Mt. Holly, N. J.
16	6 190	2½	141	1	12	Imp. New American	Dayton Globe Iron Wks. Co.	Dayton, Ohio.
1	6 490	⅛	138.7	0.2	..	Smith Wicket Gate	S. Morgan Smith Co.	York, Pa.

of this feature, at least temporarily. This is to be regarded as a setting feature rather than a runner feature. It is quite possible that there is room for an improvement here.

The authors have stated that the best results to be obtained from a horizontal setting (double unit) will always be "from 2 to 4% less than the efficiency obtained from one of the runners tested, on a vertical shaft, under the best conditions." With this statement the writer cannot agree, although he is aware of the prevalence of the opinion. He believes that by some modifications of the White hydraucone, it will be possible to derive, from a given runner, the same efficiency in horizontal as in vertical settings. The value of a rotary discharge in some such device has not yet been fully appreciated, nor realized. A rotary discharge is all that is required to increase the efficiency of a horizontal unit.

The paper names a very high efficiency obtained on a place test of the Gardner's Falls plant. Such a high value, although possible, raises the question of the kind and precision of the test. It would seem desirable to have a detailed description of this test, especially as to the determinations of power, discharge, and head.

A testing code for hydraulic turbines is now under discussion for adoption. Therefore, all matters throwing light on methods of testing would be particularly valuable at this time and the authors are certainly high in authority in this branch of hydraulics.

With reference to one of the earliest vacuum tests on a draft-tube, Mr. Gates Curtis is quoted as saying (page 1281):

"We are satisfied by experiments made with pipes leading from the top and other parts of the draft-tube attached to glass and mercury gauges that we get equal results from the wheels, as we would were they placed at the level of the tail-water. This arrangement of taking off power so far above the tail-water, thereby avoiding the annoyance and expense of bevel gearing, is proving most satisfactory in many ways. * * *"

How can the majority of the framers of the turbine test code exclude the vacuum test in the face of its obvious value? Is it because such tests frequently locate losses of 3 and 4 ft. of head in the draft apparatus?

The most pleasing and valuable parts of the authors' paper are the references to the leading men in the development of hydraulics in America and in the portraits given.

Charles W. Sherman,* M. Am. Soc. C. E. (by letter).—The part of this paper relating to the development of the turbine and describing and illustrating some of the early types of wheels, is of particular interest to the writer. Such information is difficult to obtain. Little or nothing beyond conjecture is to be found relating to the efficiency of the earlier types of wheels, and although information of this character is generally of slight significance at present, questions occasionally arise in which such data would be of service.

A number of years ago, the writer had occasion to investigate the water-power privileges affected by the diversion of the water off a small drainage area, from the head-waters of one of the tributaries of the Nashua River, for

* Cons. Engr. (Metcalf and Eddy), Boston, Mass.

the water supply of Fitchburg, Mass. The fall in this stream was considerable, and was utilized at a number of small privileges, in addition to which there were several abandoned privileges. A large percentage of the plants located on the smaller privileges were using old and comparatively inefficient types of wheels. Some of the wheels were inaccessible, and no record was available of the size, make, or other particulars.

As it is generally accepted as an axiom, that undeveloped or abandoned water-power privileges are entitled to less damages for diversion of water than fully developed privileges, it follows that partly developed privileges, or those using inefficient types of wheel, are entitled to less damages than those fully developed, with efficient power plants. Information on which to base satisfactory estimates of the efficiency of the existing plants, therefore, would have been of real significance in this investigation, but practically nothing was available. A few old wheels of the types of some of those in use, were found in mill yards, in cases where they had been superseded by better wheels, and photographs and measurements of some of them were obtained. Poor reproductions of some of these photographs have been published by the writer.* They are, however, sufficient to show in general the type of runner used in the Rose and Blake wheels. The owner of one of the mills stated that a Rose wheel tested by his father showed an efficiency of 62% when new. No information was available as to the method of the test, and the accuracy of this determination is perhaps open to question.

Among the other types of wheels found on this stream were Humphrey, Rider, and Stevens (made at Ayer, Mass.), as well as Hercules and Leffel turbines of very old types.

C. M. Allen,† M. Am. Soc. C. E. (by letter).—The authors have mentioned one or two old installations which are still being used and which give approximately the original efficiency. The writer has made a number of tests of efficiency on old installations and has found that they were still doing excellent work. For instance, two vertical 80-in. Boyden wheels in Holyoke were tested by mounting the brake on a horizontal jack-shaft. The bevel gears in both cases were of cast iron. The discharge from each wheel was carefully measured over a standard weir by A. F. Sickman, Assoc. M. Am. Soc. C. E., Hydraulic Engineer for the Holyoke Water Power Company. The highest efficiency of the wheels at full gate, which in this case includes the losses from gears and bearings, was 83.5% and 83.7%, respectively. At the time of this test, these wheels had been in operation for more than thirty years.

In another case, a Hercules wheel which had been installed in 1886, was tested thirty years later, both before and after cleaning. The brake was mounted on the horizontal jack-shaft, allowing a gear loss of 4%; this test checked the original Holyoke test of the runner. The wheel had been idle for some time and was covered with tubercles. After it had been cleaned, there was an increase of about 40% in the power.

* *Journal*, Boston Soc. of Civ. Engrs., Vol. III, p. 23.

† Prof. of Hydr. Eng., Worcester Polytechnic Inst., Worcester, Mass.

Although, occasionally, old wheels are found that are still doing as good work as they ever did, the majority are doing otherwise; altogether too many old wheels are operating, the efficiencies of which are not more than 50 per cent.

Dana M. Wood,* M. Am. Soc. C. E. (by letter).—The authors have asked for additional data of historical interest regarding old water-wheels. Fig. 35 illustrates an interesting early design of wheel. The turbine apparently occupies its original position, but the mill with its equipment is gone, and the dam, seen in the background, is in the last stages of decay. The head was apparently about 8 ft.

The turbine was set on the bed of the river and had no draft-tube, the water being discharged upward and outward. There were evidences of a 6-ft. power flume. The cast-iron gate shown, admitted the water to a spiral casing, and the shaft, with coupling on top, indicates that the turbine rests in its original position.

The plate, visible on the gate-casing, bears the inscription, "Geo. H. Jones, Little Giant Turbine, manufactured by J. C. Wilson & Co., Picton, Ontario (Canada), 1870". The power site is at Paquetteville, Que., Canada, on a small stream tributary to Hall Stream, in turn tributary to the Connecticut River.

H. A. Hageman,† M. Am. Soc. C. E. (by letter).—The authors have presented an interesting paper in reviewing the history, in the United States, of the development of the pressure type of water-turbine runner.

The mixed-flow turbine and its appurtenances have passed through trying periods in their development and have required the persistent efforts of many hydraulicians for more than fifty years to perfect the modern unit of the Francis type. In the early Nineties, turbines of the Francis type, having either vertical or horizontal shafts and equipped with one or more runners, could be obtained only for medium or low heads, low powers, and medium speeds. Prior to that period, the power was transmitted from the turbines to the driven load by gears or pulleys, and turbines were seldom direct-connected to the machine to be operated.

About that time The Cataract Construction Company, of Niagara Falls, N. Y., the predecessor of the Niagara Falls Power Company, was preparing to develop its first hydro-electric plant and was in the market for three 5 000-h. p. turbine units to operate at 250 rev. per min., under a head of 135 ft. The turbines were to be direct-connected by long shafts to alternating-current generators. The International Commission of Engineers, headed by the late Dr. Coleman Sellers, of Philadelphia, Pa., which the Company selected, invited bids from the leading manufacturers of water turbines throughout the world and finally chose a design by Faesch and Picard, of Geneva, Switzerland. The turbines selected were of the double-runner, Fourneyron type, without draft-tubes, and were built and installed by the I. P. Morris Company, of Philadelphia. The turbines and the generators of these units were unprecedented in capacity and were the admiration of the Engineering Profes-

* Engr., Stone & Webster, Inc., Boston, Mass.
† Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

Fig. 35.—Old Turbine at Paquetteville, Que., Canada.

sion. This installation was not only successful, but became the pioneer show plant in the central-station hydro-electric field, and, later, when all ten units were installed, it had a total capacity of 50 000 h. p. For many years, this plant was the largest hydro-electric station in the world. Replacement of the original wheels by single-runner Francis turbines and draft-tubes was commenced about 1910.

The successful operation of this plant gave the central hydro-electric station the necessary opening for the future design and construction of large direct-connected water-wheel generator units.

In 1899, when the present plant of the Utica Gas and Electric Company at Trenton Falls, N. Y., was constructed, the Fourneyron type of turbine was again selected. The four units installed were of the single-runner, vertical-shaft type with draft-tubes, each with a capacity of 2 000 h. p., under a 264-ft. head, at 360 rev. per min.

It seems to the writer that the two plants mentioned, containing turbines of the Fourneyron type, were largely responsible for interesting the designers of water-wheels in devoting greater effort to produce Francis turbines for much higher heads and power than could be obtained prior to 1890. This is particularly so on account of the higher part-gate efficiency of the Francis runner and also because of its better mechanical design and low maintenance cost. The result was that, about 1901, Escher, Wyss and Company, of Zurich, Switzerland, designed a turbine unit having a single runner of the Francis type, draft-tube, and vertical shaft for Plant No. 2 of the Niagara Falls Power Company. The capacity of the unit was 5 500 h. p., operating at 135-ft. head and 250 rev. per min. The completed plant contains eleven of these units and has been in successful operation for twenty years.

From 1890 to 1905, the foremost designs of water-turbine units were made in Europe, but about 1905, American designers began to produce large turbine units of the Francis type and take from the Europeans the leadership in the art. During the past fifteen years, turbine engineers in the United States have led the world in design, have produced Francis wheels of greater specific speed, head, power, and efficiency than ever before, and have developed this type of runner ahead of any other.

The general design requirements of water-turbine units are the same to-day as they have always been, but with the perfection of the electric generator, the commercial possibilities of the hydro-electric unit has increased enormously, requiring the refinement of old designs and the development of new ones to meet the new conditions brought about by competition with other forms of prime movers.

Development of water-turbine design has ever been progressive, more so during the Twentieth Century than before, and the general tendency has been to improve the efficiency of all the component parts of the unit, besides simplifying and bettering the mechanical details of construction. When the single-runner, 5 500-h. p., Francis turbines mentioned were installed at Niagara Falls, they were the largest units in the world. Since then the head, power, and speed have been greatly increased, and units for single runners with capacities

of 55 000 h. p. are in operation and similar units of 70 000 h. p. are being considered.

Thirty years ago there were few Francis turbine units operating under heads of more than 100 ft., whereas, to-day, units of this type are operating under heads as high as 800 ft., and the head or power limitation has not been reached.

Apparently, the authors are inclined to pass, almost unnoticed, the achievements of hydraulic engineers and designers of water turbines of the past twenty-five years, as represented by a group of men, many of whom are, to-day, a part of the organizations of the leading water-wheel manufacturers in the United States. Although the writer would not detract in any way from the credit which is due to the designers and engineers mentioned in the paper, he would give much credit to those engineers who, in recent years, have added inventions and improvements to the water-wheel art, and developed the Francis turbine unit as a whole, making it an efficient machine of the highest type.

Robert E. Horton,* M. Am. Soc. C. E. (by letter).—Much of the history of the development of the American type of turbine is lost. The subject is complex, as inventors in different localities developed similar ideas more or less simultaneously. Some important features of this history, however, could be recovered from a thorough study of the records of the United States Patent Office. Mr. John B. McCormick has a full set of the water-wheel patents granted by the United States Government down to 1880, which the writer has been permitted to examine. From these and from an index of water-wheel patents, Table 3, which gives a list of a few patents that represent landmarks in turbine history, has been compiled.

TABLE 3.—Important Early Water-Wheel Patents

No.	Date.	To.
.....	1804	Benj. Tyler, New Hampshire, "Wry Fly turbine".
.....	Oct. 29, 1829	Zebulon and Amasa Parker.
.....	Oct. 22, 1830	Calvin Wing, Gardiner, Me.
861	July 26, 1838	Samuel B. Howd, Geneva, N. Y.
1 876	Oct. 18, 1839	T. Rose, Windsor, N. Y.
759	May 30, 1838	N. Johnson, Triangle, N. Y.
2 599	Apr. 30, 1842	Samuel B. Howd, Arcadia, N. Y.
2 708	July 8, 1842	R. Rich.
3 153	July 3, 1843	Whitelaw Stirratt, Paisley, England.
3 510	Mar. 26, 1844	N. Johnson.
4 056	May 21, 1845	J. Leffel, Springfield, Ohio.
5 090	May 1, 1847	U. A. Boyden. (Diffuser.)
5 144	June 5, 1847	U. A. Boyden.
28 314	May 15, 1860	A. M. Swain, Lowell, Mass.
122 275	Dec. 26, 1871	M. W. and J. T. Obenchain.

The so-called "American type", large capacity, high-speed, mixed inward and axial flow turbine is undoubtedly almost wholly an American invention, but it is not the result of the labors of any one man or generation. As it stands to-day, it embodies several distinct and probably essential features; (*a*) inward flow of the water to a vortex chamber surrounding the entire runner; (*b*) a volute or scroll case, designed to give the inflowing water the proper rotary motion in the vortex chamber; (*c*) pivot guide-gates; and (*d*) a draft-tube to recover the energy remaining in the water discharged from the runner.

* Cons. Hydr. Engr., Albany, N. Y.

In view of the manner in which the water operates and the feature which most distinguishes this from all other types of turbines and water-wheels, namely, the inward flow to a vortex chamber surrounding the runner, it is suggested that the designation, "volute case vortex turbine", would better describe this machine hydraulically than the somewhat indefinite designation "American type of turbine" now in common use. It would also afford a basis of getting away from the designation "Francis turbine", a term of European origin which, for reasons hereafter stated, appears to the writer to be objectionable. If, as the writer believes, the most essential feature of this turbine was the invention of Samuel B. Howd, then following the precedent used in the case of the Fourneyron turbine, the American type should properly be called the Howd turbine. There is, however, a distinction in that in all its essential features the Fourneyron turbine stands to-day practically in the same form in which it left the hands of its inventor. The volute case vortex turbine, however, involves at least three other features generally deemed essential, which were not contributed to it by Howd. These are the volute case, pivot guide-gates, and draft-tube. The authors seem inclined to ascribe to Howd a moiety of the credit and to give Francis a large modicum. The writer has no desire to detract from the just reputation of one of America's greatest hydraulicians, the late James B. Francis, Past-President, Am. Soc. C. E. In a paper covering much the same ground as that of the authors, but in somewhat different form, the writer stated*:

"The first suggestion of a wheel of this construction appears to have been made by Poncelet in 1826. The early American inward flow wheels were, however, but a modified form of the central discharge wheel which, in turn, was evolved from the rouet. In 1838, a patent was granted to Samuel B. Howd, of Geneva, N. Y., for a simple form of inward flow wheel. Wheels made by Howd were extensively used, and were known as "United States" turbines.

"Following the general lines of the Howd wheel a turbine was designed by James B. Francis in 1849 for the Boott Cotton Mills. This wheel, constructed at the Lowell Machine Shops, was experimented upon by Francis and many similar wheels were afterward constructed on the same pattern. As the Fourneyron wheel differed from its predecessors mainly as a result of high-grade mechanical construction, so also the Francis center-vent wheel differed from the Howd and other earlier inward flow turbines. Regarding this wheel, Francis says in 'Lowell Hydraulic Experiments': 'In the design for the Boott wheel the writer has so modified the form and arrangement of the whole as to produce a wheel essentially different from the Howd wheel as above described, although it may possibly be technically covered by the patent for that wheel'. As above stated, impartial comparison would hardly leave any thing to the credit of Francis in the invention of this wheel, although the construction was so much improved as to make it, as he says, 'essentially a different wheel'. In fact, the progress of the last century in industry has been very largely the result of improved mechanical construction of machines, the principles of operation of which have been known from an earlier period. The center-vent wheel was quickly displaced in America by the inward and downward flow American stock pattern type of turbine, and it is probable there was not a single wheel of the Francis type in use in this country for nearly half a century. In the meantime, the simple inward flow turbine was extensively used on the European continent under the name of the Francis

* "The Turbine Water Wheel as a Prime Mover", *Bulletin*, Clarkson Inst. of Technology, Potsdam, N. Y., Vol. 7, No. 1 (January, 1910).

turbine. To meet the demand for a compact, large capacity, high-head water-wheel, it is being reintroduced into the United States, following the European designs, and it is now known here as the 'Francis turbine', although it has the volute or spiral case developed abroad by Schiele and commonly used in America with crude central discharge wheels. It has the pivot gates of Fincke and probably resembles the original Boott turbine of Francis less than that resembled the cruder Howd wheel. It is quite certain that this type of wheel is to be the hydraulic prime mover of the future."

The writer has not experienced any important subsequent change of view in this matter, except to come to the conclusion that the work of Howd was of relatively greater, and that of Francis of less, importance in this connection than the foregoing quotation indicates.

Referring to the Howd wheel patented July 26th, 1838, Francis says, "a wheel similar in its essential features was proposed in France in 1826 by Poncelet".* The Poncelet turbine referred to is illustrated and fully described by Weisbach.† He says: "The first idea of an inward flow tangential wheel is due to Poncelet". This wheel, as shown by the illustration and as intimated by the designation, was fed by a tangential spout, the water entering on a small arc of the circumference only. It did not embody the principle of the vortex chamber which was that the water surrounding the wheel should rotate at such velocity that it could enter the runner without shock. That is the essence of the Howd type of turbine and of the modern scroll case vortex turbine. The writer has been unable to find any precedent in the historical or patent literature of hydraulic motors for an inward flow turbine fed around the entire circumference, antecedent to Howd's patent of 1838. In order to get a better perspective of the situation attending Howd's invention, it is worth while to consider the opinions and practices of the times in water-wheel matters.

At that date, there were no great firms engaged in the manufacture of water-wheels. It was the usual practice for the millwright to build the water-wheels for the mill at which he was employed, and as a rule the old-time millwrights were exceedingly staunch in their opinions and jealous of their prerogatives. On the one hand, the dominant water motors were of the horizontal, overshot, and breast types; and on the other, wooden spout-fed flutter, tub, and scroll case, central discharge wheels. Fourneyron had invented the outward flow turbine in Europe a few years previously, had brought it to a high state of mechanical excellence and efficiency, and its fame had spread the world over. Simultaneously with Howd's invention, Morin's classical experiments on the Fourneyron turbine were being conducted, the results of which were shortly translated by Ellwood Morris and published in the *Journal* of the Franklin Institute. Almost immediately following Fourneyron's invention, crude outward flow turbines had been brought into use in the United States to meet the demand for a relatively high-speed water-wheel to drive the vertical English gate and muley saws in the mills of the pioneers. Calvin Wing had patented such an outward flow turbine without guides in 1830.

* "Lowell Hydraulic Experiments", p. 61.

† Du Bois, "Mechanics of Engineering", Vol. II, Hydraulics and Hydraulic Motors, p. 364.

The use of such wheels spread with great rapidity, despite the limited means for disseminating information in those days, and during the period from 1838 to 1850, when Howd's work was performed, numerous patents were issued on cast-iron, outward flow turbines identical in principle with that of Calvin Wing and including the well known Rose, Johnson, and Rich turbines. The tendency of the times was strongly toward the use of outward flow turbine wheels. The average millwright had a sort of superstitious belief in the magical efficacy of centrifugal force, which it was well known augmented the discharge and increased the power of outward flow wheels. In the light of these circumstances, the writer is inspired with a feeling of great respect for the moral courage and creative imagination of Howd who undertook, practically without precedent, and in the face of such well established opposition, to develop and commercialize a turbine water-wheel acting on directly the opposite principle by having the water flow inward instead of outward. Unfortunately, little is known or can apparently be learned about the life and work of Howd. When his first patent was granted he was a resident of Geneva, N. Y. It so happened that some of the writer's ancestors were pioner millwrights and water-wheel builders in the same locality and at the same date. Diligent inquiry, however, has failed to reveal anything regarding Howd.

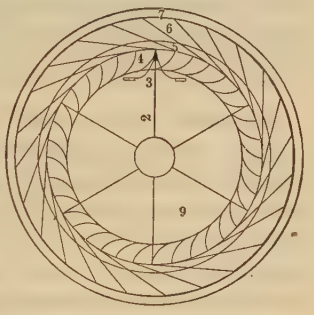


FIG. 36.

It is entirely true, as the authors state, that Howd did not himself manufacture turbines, but sold patent rights with the expectation that the wheels would be constructed by millwrights, at least the writer has not been able to find any evidence to the contrary. In the "American Miller and Millwright's Assistant", by William Carter Hughes, 1851, appears a diagrammatic outline entitled "Howd's Patent Direct Action Water Wheel", which is here reproduced as Fig. 36. In the same book (page 57), is an article purporting to have been written by Howd, which contains the following:

"This is a wheel which, when properly located, is admirably adapted for mills of all kinds, working the water on the tourbillon principle, being the whirling vortex or better known as reaction principle. Its superiority over the old reaction wheel consists in applying the water on the wheel at the verge and discharging it at the center."

This quotation is offered in view of the author's statement that "it seems doubtful whether he appreciated the value of centripetal flow." Hughes' book also gives "directions for making the several parts of Howd's latest improved water-wheel and setting it up." The text indicates that it was presumed that the runner crown would be made of plank, and directions were included for fitting the runner to a wooden shaft, octagonal wooden shafts being the usual type of construction in those days. In the few succeeding years, to the time when James B. Francis, with keen insight, recognized the merits of the Howd wheel, there were great improvements in facilities for mechanical operations.

It is not surprising that, with his own great ingenuity and the mechanical resources of the Locks and Canals at his disposal, Francis should have been able to have built for him by skilled mechanics, wheels of this type of better construction than those of the ordinary millwright, but a comparison of the diagram of the Howd wheel as constructed by Francis for the Boott Cotton Mills (Fig. 37) shows the layout of the guides and runner closely resembling Howd's diagram, Fig. 36. There was no change or improvement whatever in the principle of operation.

FIG. 37.

The writer has a copy of Howd's Patent No. 861, dated July 26th, 1838. The patent drawings include a plan of the layout of the runner and guides practically identical with that shown in Fig. 36. The claim is of particular interest. It is as follows:

"I claim as my invention—the application of the water upon the outside of the wheel and operating upon the principle of reaction by discharging inwardly on a wheel constructed and combined so as to operate as above described with the spouts or shoots giving the water a direction with the motion of the wheel applied to a reacting wheel as aforesaid."

Inasmuch as the writer has not found any precedent for an inward flow reaction turbine supplied with water around the entire circumference antecedent to Howd's patent, it appears that credit for the invention of this important type of prime mover clearly belongs to Howd, and that the runner as he originally conceived it, with the possible exception of the use of curved instead of straight guide vanes, was identical with the runner from which Francis and others designed or constructed similar wheels.

The writer has felt impelled to take up this matter in view of the fact that the merit of Howd's invention has been in the past, and appears likely to be in the future, greatly dimmed by the reputation of James B. Francis. In view of the fact that the turbine, next to if not indeed transcending the steam engine, is one of the greatest of human inventions and that the high-head turbine of to-day embodies the principle of vortex circumferential feed to the runner precisely as it left the hands of Howd, it has seemed proper to suggest that Samuel B. Howd is entitled to a place in the Hall of Fame as one of America's foremost inventors. This discussion is given solely in the spirit of "rendering unto Cæsar the things that are Cæsar's", and as stated, not with any desire to belittle the reputation or standing of James B. Francis.

In view of the preceding and for other reasons, the writer does not feel willing to accept the "Family Tree" of the mixed-flow turbine as given by the authors. Under early American turbines are included the spiral, Rich, and Rose wheels. These, it is true, were early American turbines, but they were not in any sense antecedents of the volute vortex turbine. Again, the American, Hercules, Victor, and Sampson turbines were offshoots of the Obenchain and McCormick wheels, which, in turn, were direct predecessors of the modern high-speed runner. As a matter of fact, the essential features of the volute-vortex turbine of to-day were more or less separately, but simultaneously developed and cannot be properly ascribed to a common ancestral tree. Subject to revision, the direct antecedents of the principal features of the modern

volute-vortex turbine appear to the writer to have been substantially as set forth in Table 4. The draft-tube has not been included, as it appears to have been the independent creation of Jonval and has received little subsequent change or improvement, except the outlet cone, undoubtedly suggested by Boyden's diffuser, and built at an early date by Swain and others, and more recently by Moody.

TABLE 4.—Prototypes of Modern Volute-Vortex Turbines, Showing the Parallel Development of Various Features.

Development of inward and mixed inward downward flow principle.	Development of the scroll-vortex chamber.	Development of pivot guide gates.
(1) Scroll or Spout Feed; no Guides: (*a*) No Reaction: Primitive wooden scroll central discharge water-wheels. (Circ. 1820) (*b*) With Reaction: Poncelet Turbine (1825) (2) Full Circumferential Feed with Reaction and Guides: Samuel B. Howd. 1838. (3) Increased Capacity and Speed, Howd Type: Francis-Howd 1847 Swain 1860 (4) High-Speed Type: Obenchain—Little Giant 1873 John B. McCormick Circ. 1870-1885. (5) Propeller Type: Austin wheel. Screw current meter. Marine propeller. Nagler high-speed runner.	(1) Vertical Spout-Fed Flutter Wheel (Medieval). (2) Primitive Wooden Scroll Case Central Discharge (Circ. 1820). (3) Parker Scroll Case (Pat. 1829). (4) Iron Scroll Case Reaction Turbines without Guides: Tyler, Reynolds etc., etc. (Circ. 1850) (5) Scroll Case Reaction Turbine with Guides: Warren (Pat. 1860) Early Risdon. Schiele Turbine. (6) Partial Vortex Scroll Case with Pivot Guides: Thomson Vortex Turbine (Circ. 1870)	(1) Early Leffel Turbine 1845-1860. (2) Thomson Vortex Turbine (Circ. 1870). (3) The Fincke Pivot Gate, developed recently in Europe.

The authors, quoting Pfarr, credit J. M. Voith, of Heidenheim, with being the first to build an iron, spiral-cased, wicket-gate turbine, thus incorporating in one machine all the essential features of the latest volute-vortex turbine. Wooden, scroll-case, central-discharge wheels with flat vanes were in common use in Central and Northern New York in the early part of the Nineteenth Century, quite certainly as early as 1820. It is probable that they were originated simply as an improvement or evolution of the vertical spout-fed flutter wheel, which has been traced back to Medieval times. From about 1850 to 1880, scroll case wheels without guides, including the Reynolds, Tyler, and Carley, were in common use. As far as the writer can determine, the Warren wheel, referred to by the authors, was the first scroll case reaction turbine with guides, which appears to have been patented in 1860. It was probably somewhat later that James B. Thomson invented his vortex turbine (Fig. 38). This wheel had a volute case, the manner of feed resembling somewhat more closely that of the modern turbine in a concrete volute chamber than that of the high-head volute-vortex turbine in an iron scroll case. It had pivot guides, however, and appears clearly to antedate Voith's claim of being the first turbine which incorporated all the essential features of the modern volute-vortex turbine.

The earliest use of pivot guide-gates is somewhat in doubt, and possibly originated with the early Leffel wheel. The modern type of pivot guide-gate is a recent European invention, commonly ascribed to Fincke.

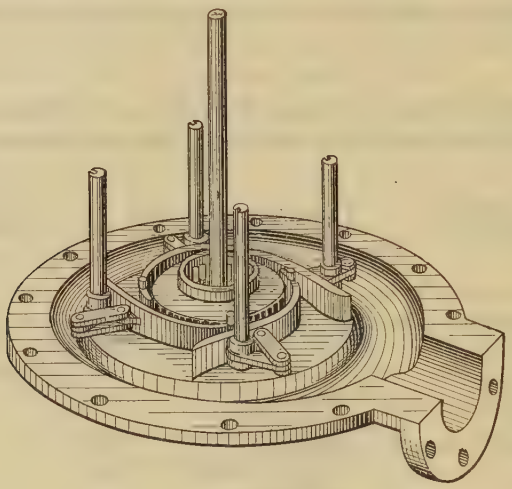


FIG. 38.

With reference to the propeller type of runner as developed by Mr. Forrest Nagler, it may be noted that this runner differs from that of the volute-vortex turbine in several particulars:

(1).—It is more essentially a downward than an inward flow runner.

(2).—The crown plate and rim band are dispensed with and the runner vanes project radially and are otherwise unsupported from the hub as in a screw propeller.

(3).—The plane of each runner vane is nearly at right angles to the axis, thus giving a runner of high speed of rotation.

(4).—The number of vanes is much more limited than in the case of the vortex type of runner.

The first of these features, namely, axial flow, has been common since the days of Jonval. The omission of crown and rim bands, although, perhaps, not very essential, has been not uncommon in other water-wheels, beginning with the vertical flutter, tub, and scroll central discharge types.

TABLE 5.

Name.	Page.	Crown plate.	Rim band.
Blackstone	188	Yes	No
Economy	194	No	No
IXL Compound	194	No	No
Mullikin	200	Yes	No
Coleman	216	Yes	No
Walsh	335	Yes	No
Mercer	355	Yes	No

TABLE 6.—Tests of S. Reynold's Central Discharge Wheel by Charles J. Rhodes, Oswego, N. Y., August 23d, 1873.*

Number of experiment.	Duration, in minutes.	Total head and fall, in feet and inches.	Effective head when wheel out of water from head to center inlet, in feet and inches.	Effective head, in feet and decimals.	Velocity due to head, in feet and decimals.	Revolutions of wheel.	Revolutions per minute.	Velocity of circumference diameter = 10¼ in.	Ratio.	Depth of water in reservoir, in inches. Size = 64⅛ sq. ft.	Quantity of water, in cubic feet.	Quantity if flowing through inlet without contraction or obstruction. Size of inlet = 6.127 sq. in.	Ratio.	Weight on brake, in ounces.	Length of brake, in inches.	Useful effect in pounds, raised 1 ft. per min.	Theoretic power.	Ratio or percentage of effect.
1	10	4- 5¾	4-3	4.25	16.50	2 930	293	13.1	80	33	177.8	421	42	20	15	2 876	4 723	60.8
2	11	4- 4¾	4-2	4.16+	16.43	3 100	282	12.6	77	40⅞	220.0	458	48	23	"	3 184	5 206	61.1
3	10	4- 4¾	4-1 15/16	4.16+	16.43	2 250	255	11.4	70	38⅝	207.8	413	50	28½	"	3 567	5 411	65.9
4	10	3-10½	3-7 11/16	3.64	15.26	3 753	375	16.8	110	24⅜	131.3	398.5	37.7	0	"	0	2 987	0
5	10	3-10½	3-7 11/16	3.64	15.26	2 558	256	11.46	74	38⅝	206.8	398.5	53	28½	"	3 577	4 705	76.0
6	10	3- 9	3-5 11/16	3.47	14.90	2 218	222	9.9	66	39	209.8	380	55	32	"	3 487	4 550	76.6
7	10	3- 8	3-5 3/16	3.43	14.80	1 812	181.2	8.3	56	39 11/16	213.5	378	56+	37	"	3 274	4 578	71.5
8	10	3- 8	3-5 3/16	3.43	14.80	1 413	141.3	6.3	42	40 1/16	218.5	378	57.8	41	"	2 837	4 684	60.4
9	10	3- 7¾	3-4 15/16	3.41	14.80	.....	.....	....	0	41⅜	225.9	378	59.5	58	"	0	4 814	0
10	9	3- 8½	3-5 11/16	3.47	14.90	2 080	231	10.3	70	34⅛	183.6	342	53.6	30	"	3 401	4 424	76.8
11	10	3- 3½	3-3½	3.29	14.50	2 023	202.3	9	62	36⅝	197.0	370	53	30	"	2 974	4 063	73.2
12	10	3- 3	3-3	3.25	14.40	2 088	209	9.3	64½	35⅞	192.9	368	52.4	27½	"	2 820	3 918	72.0
13	10	3- 5	3-5	3.41+	14.80	2 375	237.5	10.6	71	35 7/10	190.7	378	50.4	25	"	2 914	4 198	69.4
14	10	3- 5¼	3-5¼	3.44−	14.80	1 933	193.3	8.6	58	38⅛	205.0	378	54.2	32	"	3 036	4 407	68.8
15	10	3- 5	3-5	3.41	14.80	1 760 / 1 785	176 / 182.5	8	54½	38⅝	206.7	378	54.6	35	"	3 004	4 405	68.1
16	5	3- 5	3-5	3.41	14.80	0	0	0	0	20½	110.0	189	58.2	56	"	0	4 676	0
17	5	3- 7	3-7	3.58	15.10	1 858	378	16.8	111	13⅜	71.8	192	37.4	0	"	0	3 213	0

* Copied from originals on file with the Oswego Canal Company.

Emerson* illustrates a goodly number of water-wheel runners with either the crown plate or rim band, or both, omitted, including those shown in Table 5.

The writer does not feel that the Truax turbine, described by the authors, is particularly fitting as an illustration of the antecedents of the propeller type of runner. It appears clearly from the manufacturer's circular and illustration that this was essentially the same as many common types of Jonval turbine. It had a rim band, four buckets, and a larger vane angle than the propeller type. The Austin wheel, although also of the Jonval type, with rim band, had only two vanes which were set at a slight angle and thus gave to that wheel the most essential feature of the propeller type of runner, namely, a very high specific speed.

With reference to the early testing of turbine water-wheels, it is worth while to note that, in 1873, Charles Rhodes, Agent of the Oswego Canal Company, a corporation leasing water on the Oswego River in a manner similar to the practice at Lowell, Lawrence, and Holyoke, conducted an extensive series of tests of the types of water-wheels then commonly in use. Rhodes was an engineer familiar with the work of James B. Francis, which he apparently followed closely in conducting these tests. A sample sheet of the results of one of Rhodes' tests is reproduced in Table 6, not only because of its historical interest, but because it contains apparently authentic results giving the efficiency of one of the iron scroll case reaction types of turbines without guides, similar to the Tyler wheel, which was in common use in the period about 1850 to 1880, particularly for driving buhr stones in flour mills.

Forrest Nagler,† Esq. (by letter).—This paper fills a long felt want in emphasizing the fact that modern hydraulic turbine forms not only have reached their highest development in America, but actually originated here. This will probably come as a shock to the large number of American engineers, and particularly to editors of technical papers, who have been too prone to accept the widely spread doctrine of European supremacy in all things technical, particularly as applied to water power. It is fortunate that so rich a collection of data as that which exists at Lowell has been opened to the public in this manner by engineers so familiar with early development work and the men who conducted it, as well as with the modern tendencies in turbine design.

The paper presents new information on the contention previously made by the writer before the American Society of Mechanical Engineers, to the effect that modern reaction wheels should have been called "Swain" instead of "Francis" runners, and only serves to strengthen the belief that the change is in order. The real change from the straight radial outward or inward flow type, as used by Fourneyron and Francis, is that which extends the buckets around the annular elbow. This change was first made by Swain. It resulted in doubling or tripling previous specific speeds and made similar increases in horse-power, as is evidenced by Fig. 21. As embodied in the Swain and

* "Hydrodynamics", 1894 edition.

† Hydr. Engr., Allis-Chalmers Mfg. Co., Milwaukee, Wis.

Risdon turbines, this type also permitted the greatest single increase in efficiency that appears in the history of turbine development. Apparently, Francis never developed or used the mixed-flow runner bucket which is characteristic of all modern reaction wheels, but was associated with it only through descriptive articles or tests of Swain's products. As this type permitted the greatest advance experienced up to the time of its development and has since dominated the entire field, the writer suggests that it is strictly in order to designate modern reaction wheels as being of the Swain type. He believes the "Family Tree" shown on page 1270, should be re-arranged with the name, Swain, placed parallel with the name, Francis, rather than being in series with it, which re-arrangement would permit the name, Swain, to dominate the entire field of mixed-flow turbines, including all the earlier types mentioned and leading up to the modern high-speed types. Comments by Clemens Herschel, Past-President, Am. Soc. C. E., on the point would be particularly interesting.

TABLE 7.—Test of Green Mountain Four-Blade Propeller Turbine.
Runaway Speed = 1 232 rev. per min. Head = 12.70 ft. Discharge = 8.085 sec-ft.
Water Temperature = 75.56° Fahr.

Experiment No.	Speed, in revolutions per minute.	Head, in feet.	Discharge, in second-feet.	Horse-power.	Efficiency.	Specific speed.
1	244.3	12.90	4.91	1.940	27.0%	13.90
2	288.8	12.90	4.99	2.283	30.6	17.70
3	333.0	12.90	5.085	2.497	33.5	21.50
4	377.5	12.90	5.27	2.705	35.1	25.40
5	422.0	12.89	5.355	2.925	37.7	29.60
6	466.2	12.89	5.475	3.023	37.8	33.20
7	510.8	12.88	5.58	2.642	32.4	34.20
8	532.2	12.88	5.65	2.520	28.8	34.60
9	488.5	12.89	5.51	3.022	37.5	34.70
10	444.0	12.89	5.45	2.971	37.3	31.40
11	400.0	12.90	5.28	2.810	36.3	28.10
12	355.5	12.90	5.18	2.60	34.3	23.40
13	311.0	12.90	5.05	2.38	32.2	19.60
14	266.0	12.90	4.95	2.11	29.5	15.70

The Green Mountain turbine shown on page 1264 might be considered to be a forerunner of the type of turbine developed by the writer. It is believed, however, that it failed not so much by reason of lack of necessity for high speed as by reason of lack of recognition of the hydraulic principles involved. The use of a rim, together with a large blade area, introduces such additional friction as to form a brake, so that the speed is greatly reduced. Tests of two of these old wheels—one of the Green Mountain type and the other of the Austin two-blade type—shown in Figs. 39 and 40, indicate that the obtained efficiencies and specific speeds are so far below those attainable with the writer's designs as to make the earlier forms uncommercial. The data given on page 1264 indicate a discharge of approximately 15 cu. ft. per sec. under a head of 1 ft. This means a velocity of from 3 to 6 ft. per sec., depending on the angle of whirl. These velocities fix the discharge loss at between 12 and 55% of the total head. The actual discharge loss is probably always in excess of 30%, so that if all other features were perfect, the efficiency could not be

more than 70 per cent. It is necessarily much lower, as the blades are practically helical in form, the Chase runner shown in Fig. 17 being a possible exception. The helical form of blade is not best suited for the development of high efficiency, as it does not take fullest advantage of the suction principle to which the writer has paid great attention in his designs. Mr. Horton's statement, quoted on page 1265, to the effect that the efficiency of these flat-angle screw runners is very low, is confirmed by the writer's tests of models of these original wheels.

Complete data of these tests are given in Table 7 for the Green Mountain or Truax propeller shown in Fig. 16, and in Table 8, for the Austin wheel referred to by Mr. Horton. The former is a four-blade wheel substantially duplicating that shown in Fig. 16, but without the saw-teeth, and the latter is a two-blade wheel, otherwise substantially similar in profile to the Green Mountain.

TABLE 8.—Test of Austin Two-Blade Propeller Turbine.

Runaway Speed = 615 rev. per min. Head = 12.92 ft. Discharge = 2.13 sec-ft.
Water Temperature = 76.1° Fahr.

Experiment No.	Speed, revolutions per minute.	Head, in feet.	Discharge, in second-feet.	Horse-power.	Efficiency.	Specific speed.
1	222.0	12.96	1.70	.694	27.7	7.15
2	244.2	12.96	1.71	.760	30.2	8.68
3	266.4	12.96	1.74	.825	32.3	9.91
4	288.6	12.96	1.755	.885	34.3	11.05
5	310.8	12.96	1.765	.942	36.3	12.28
6	333.0	12.96	1.80	.970	36.7	13.34
7	355.2	12.96	1.82	1.035	38.7	14.72
8	377.4	12.96	1.85	1.077	39.5	15.90
9	399.6	12.96	1.85	1.140	41.9	17.33
10	421.8	12.96	1.89	1.165	42.0	18.50
11	444.0	12.96	1.90	1.215	43.5	19.95
12	488.4	12.96	1.93	1.265	44.6	22.16
13	466.2	12.95	1.93	1.262	44.5	21.30
13	488.4	12.95	1.96	1.248	43.4	22.30
14	510.6	12.94	1.98	.940	32.3	20.20
15	522.0	12.94	2.00	.698	23.7	17.80
16	499.0	12.95	1.97	1.048	36.2	20.80

Both these wheels were tested in a setting closely approximating that shown in Fig. 16, that is, without any guide case and without any draft-tube. No data are available as to whether these wheels were operated slightly above tail-water level or "drowned", although the indications are that the tail-water level is usually slightly above the bottom of the wheel. It was found that the efficiency of these wheels is slightly greater if they are not "drowned", provided the head is measured from the head-water level to the bottom of the wheel rather than to the tail-water level. In all the tests shown in Tables 7 and 8, the head was taken to the bottom of the wheel, whether the tail-water level was at this point or lower. During the tests every precaution and care in design, installation, and operation that is practised in modern testing was observed. The arrangement in every way was the equal of that used in modern commercial wheels and as compared to the original setting of these old wheels, was considerably superior, as there was no obstructing spider or step support below

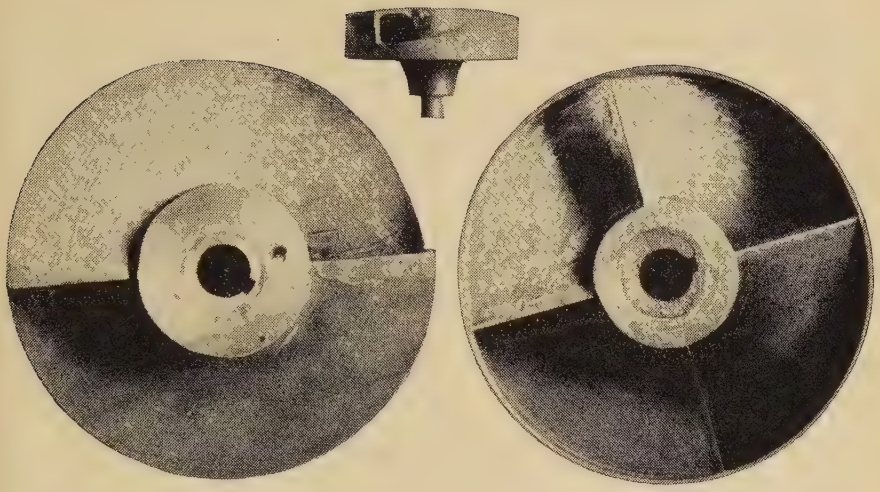


FIG. 39.—TEST RUNNERS OF AUSTIN TWO-BLADE TYPE AND GREEN MOUNTAIN TYPE.

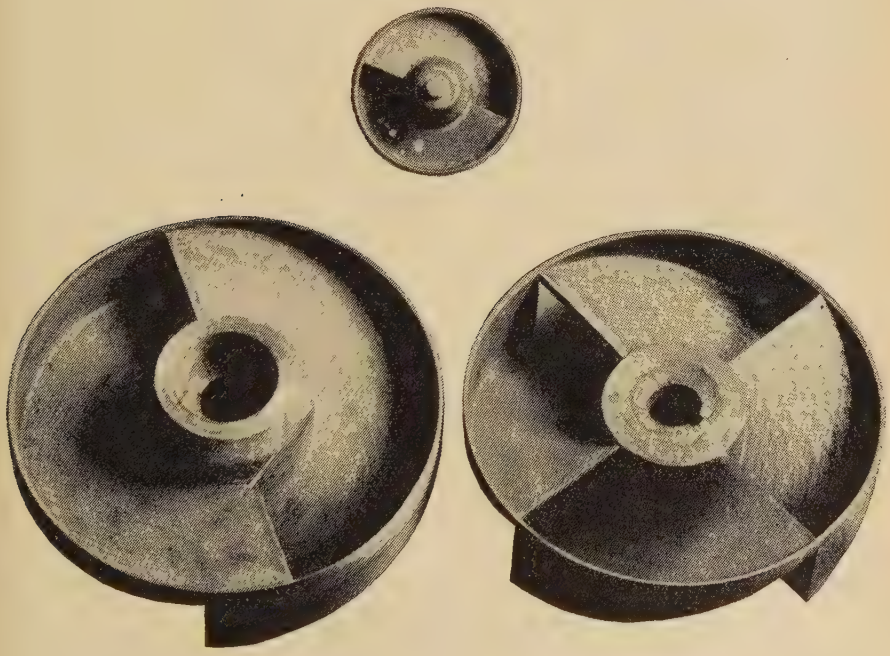


FIG. 40.—TEST RUNNERS OF AUSTIN TWO-BLADE TYPE AND GREEN MOUNTAIN TYPE.

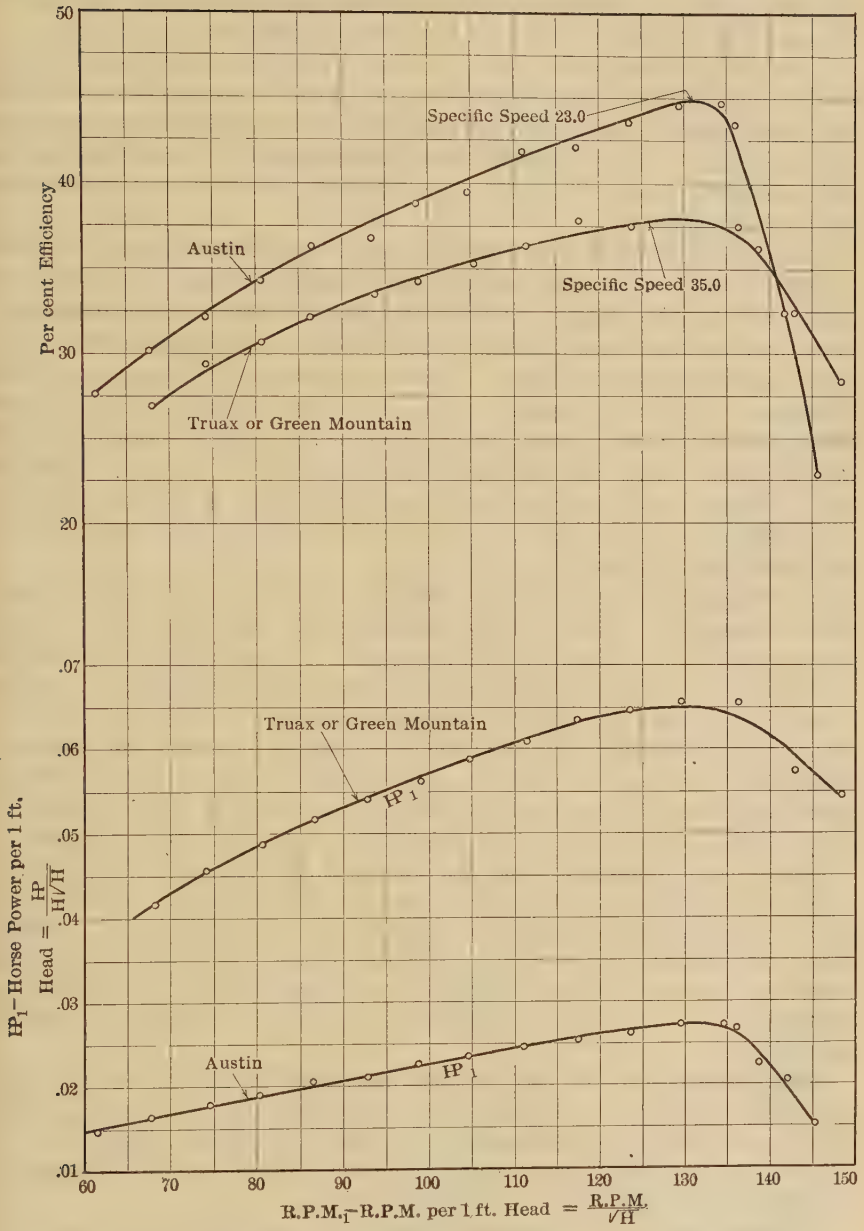

Specific Speed 23.0
Austin
Specific Speed 35.0
Truax or Green Mountain
Per cent Efficiency
50
40
30
20
Truax or Green Mountain
HP_1
.07
.06
.05
.04
.03
Austin
HP_1
.02
.01
HP_1–Horse Power per 1 ft. Head = $\frac{HP}{H\sqrt{H}}$
60
70
80
90
100
110
120
130
140
150
R.P.M.$_1$–R.P.M. per 1 ft. Head = $\frac{R.P.M.}{\sqrt{H}}$

FIG. 41.

the wheel to interfere with free discharge. This was accomplished by carrying the thrust load on a ball-bearing at the top of the shaft near the brake pulley. For convenience, the efficiency and horse-power curves are shown on Fig. 41, the power being given on a basis of 1-ft. head for both wheels.

Tests of the Chase wheel, shown in Fig. 17, indicate specific speeds less than 100, even with the most favorable setting of guide case and draft-tube and show efficiencies of considerably less than 70 per cent. Eliminating the draft-tube and using any type of guide case of which record is available as having been used with this wheel, the tests show efficiencies less than 50% and specific speeds of about 40.

All these tests were made on 10-in. runners, under heads varying from 10 to 15 ft. The runners were made from original drawings and designs or copied from existing wheels. All these models are proffered for inspecting, measuring, checking with originals, or re-testing, to any individual or committee authorized by the Society. Analysis of these tests shows a striking discrepancy between the assumptions made by the authors in Fig. 21 and Table 1, in which they assume an efficiency of 60% and a specific speed of 125 for wheels of the propeller type, based, apparently, on trade literature data. The previously mentioned tables and curves indicate that the efficiency is considerably less than 50% and the specific speed less than one-third the figure given by the authors.

The data given herewith present striking comments on the author's statement, on page 1238, to the effect that "a wheel of the high-speed propeller type was patented and on the market fifty years ago". It is granted that the wheels to which they refer had radial blades few in number, but the high-speed feature is questioned. They required the addition of a suitable guide case, the addition of suitable diffusers or draft-tubes, the elimination of the rim, approximately 50% reduction of blade area, and a considerable modification of the blade angles from the substantially helical form originally used, in order to permit of the possibility of securing high speed as it is understood at present.

It may be of interest to note that all the early designs of wheels of the propeller type discussed by the authors have been considered by the officials of the U. S. Patent Office in their investigations of the Nagler design. These officials have expressed the opinion that these modern designs of high-speed runners and turbine settings, although they include certain old elements, do produce new and useful results and decided advances in the field of hydraulic engineering.

The horse-power of the last runner, designated as the modern propeller type, Fig. 21, has been increased about 50% above the figure given. The specific speeds have been increased more than 50 per cent. Table 1 is interesting, the most significant features being the high efficiencies and increases of power made by the Swain and Risdon turbines.

In connection with the section on draft-tubes, no distinction is made between the form of modern concentric tube, the measure of the efficiency of which is in the closeness with which it approaches the form of natural hydraucone, developed and defined by Mr. W. M. White, and the old form,

which merely provided for transition in a chamber from axial to radial flow. It is the development of the proper form of chamber, that has permitted the attainment of results, impossible with any of the chambers of which Figs. 32 and 33 are typical.

GEORGE A. ORROK,* M. AM. SOC. C. E.—The speaker is particularly interested in Table 1 which shows that forty-five years ago runner efficiencies, as tested at Holyoke, had reached a figure in excess of 90 per cent. In 1920, efficiencies from 92 to 93% were common, and runners built in small country shops had given these test results. The speaker believes that the published figures of more than 95% for one of the largest runners and draft-tubes tested in the field, is the present-day limit of attainment, and he is inclined to attribute this more to the cleverness of the runner designer than to the saving due to a perfectly designed draft-tube.

F. W. SCHEIDENHELM,† M. AM. SOC. C. E. (by letter).—Some of the efficiencies recorded in Table 1 seem truly remarkable. Great credit is due such men as the designers and makers of the Risdon wheels of 1873 and the "Hercules" wheel of 1876. It is more than likely that the tests of the present Holyoke flume are somewhat more severe in results than the earlier tests made there, thus accounting for the fact that to the better known wheels of the period extending from the construction of the present flume in 1881 to the end of the century are imputed lower efficiencies than the premier wheels of 1873-76. Even if this is true, the results shown in Table 1 for the period prior to 1900 would indicate on the face retrogression in the early years of the present century. However, such is probably not the case.

Undoubtedly, the most reliable index of performance of the water-wheels of a given type is the extent to which the manufacturer is willing to guarantee that performance. Still the progress of guaranties of efficiency has not been compatible with so small an increase in attained efficiency as one might infer from Table 1 to have taken place since the dates there shown. Nowadays, a test efficiency approximating 94% is fully as rare as one of 90% is reported to have been in the Seventies; nevertheless, the difference between present-day guaranties and those of, say, 1900, are considerably greater than 4 per cent. At present, there is such a thing as a guaranteed efficiency of 90%; whereas, as late as 1905, guaranties rarely exceeded 80 per cent.

A possible explanation is that the highest efficiencies shown in Table 1 represent exceptional, not average, wheels; that the then existing state of the art did not afford types the individual wheels of which were consistently of high efficiency. However, one of the outstanding characteristics of the present state of the art is the consistency which exists in this very respect. If this was not the case, the manufacturer would not be so ready to make guaranties of high efficiency and the purchaser would have to pay a considerable insurance premium to compensate the manufacturer for failures to attain the guaranteed mark. As a matter of fact, failures are now rare.

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† Cons. Engr. (Mead & Scheidenhelm), New York City.

Perhaps a further explanation lies in the extensive use prior to 1900, and even later, of "stock" wheels, whereas plants of larger capacity justify special runner development wherever the controlling conditions so indicate.

In short, much as credit may be due to the pioneers for the attainment of certain exceptional efficiencies, to the present generation belongs the credit of making efficiencies around 90% reliably available.

Again, the authors apparently are of the opinion that the results of hydraulic turbine tests at the Holyoke flume are accurate and reliable. It seems, however, that of even greater significance from the practical standpoint is the fact that the field tests of modern reaction turbines of great capacity generally yield efficiencies higher than those obtained in tests of the smaller homologous runners at Holyoke or elsewhere. This is likely the rule only for cases where the head under actual operating conditions is materially greater than the head under test conditions and where, therefore, a constant friction loss is a larger proportion of the power under the lower test head than of the power under the higher operating head. Nevertheless, the writer must confess that, until about ten years ago, he, like many others, was inclined to be skeptical of laboratory tests as a direct measure of the efficiency to be expected from the full-size runner in place. On the contrary, the fact can no longer be successfully controverted that, unlike laboratory results in general, laboratory tests of turbine runners show at least as low efficiencies as field tests. A difference of 2% is by no means unusual.

The development of the hydraulic turbine has been largely empirical. Time and again theory has not stepped in or explained progress until after experimentation, such as marked the "cut-and-try" period described by the authors, had made that progress a reality. The present-day development seems to be no exception.

Thus, the increase in unit or specific speed, under the incentive of lower generator costs, has been taking place gradually. The step to a still higher specific speed for application in a given problem of the moment seems to be hindered, not by theoretical considerations, but only by lack of precedent. The writer believes that the development of reaction wheels has not yet reached the point where one can determine the maximum specific speed which will ultimately be practicable for a given head.

Similarly, the reaction turbine has been applied successively and successfully to higher and higher heads. About 1912, the feasibility of application to heads of approximately 600 ft. was a matter awaiting confirmation by practical operation. At present, the reaction turbine has been used with heads of more than 800 ft. The ultimate upper limit seems to await determination by practical test rather than theory.

Recently, the writer has had occasion to refer to Weisbach's "Mechanics of Machinery and Engineering", 1846-47, as translated from the German and published in Philadelphia, 1848-49. The two volumes of this work had recently been given the writer from the library of the late Frederic P. Stearns, Past-President, Am. Soc. C. E., and, as now appears by signatures in the volumes,

must originally have belonged to "U. A. Boyden". This was undoubtedly Uriah A. Boyden, the "father" of American mixed-flow hydraulic turbine design.

By reference to the prefatory matter in the second volume of Weisbach, which deals at length with water-wheels and turbines, it appears evident that in 1849 both the English translator and the American editor considered that this was "the only theoretical treatise on water power of the least practical value hitherto printed in the English language". Inasmuch as Boyden designed his first water-wheels five years or more before he could have read what Weisbach had to say on the subject, it would seem that in his case, also, experimentation and common-sense design preceded theoretical analyses. Moreover, it is possible that his subsequent scientific analyses, especially of the flow of water through the turbine, found inspiration, if not basis, in Weisbach's detailed treatment of the "theory of reaction turbines."* Certainly, after reading Weisbach,† Boyden would have had no excuse for not appreciating the fact that in the language of the authors (page 1254), "some energy remained in the water at the time of its discharge from the runner."

Jay M. Whitham,‡ Esq. (by letter).—This paper is a noteworthy and interesting contribution on the subject, and preserves, in compact form, much of the early history of turbine development. The paper shows, and deservedly so, great admiration for the notable work performed by the late James B. Francis. Although the Howd wheel is mentioned, the credit for its development is given to Mr. Francis. This, the writer believes, is not fully warranted. It is to be regretted that so little is known of the achievements of Howd. To the writer, the modern mixed-flow turbine is Howd's "grown-up child", and it possesses his characteristics.

Gardner S. Williams,§ M. Am. Soc. C. E. (by letter).—The authors have presented a very interesting history of the development of the American turbine, a subject which previously has not been adequately treated. Several years ago, the writer made investigations along the same line as those of the authors, and recalls a few points that are not included in the paper. As to the so-called wicket gate, it appears that a patent was issued for this device to John Temple, February 8th, 1859, and by him assigned to Stout, Mills and Temple, the predecessors of the Dayton Globe Iron Works, who manufactured it with the American wheel at Middletown, Ohio, until 1863, when the works were removed to Dayton. It has always been a characteristic of the "American Turbine." How it happened that Leffel was permitted to use it, had been a mystery to the writer until he saw the authors' statement that the gate had also been patented by Elijah Roberts five years earlier.

The Leffel Wheel.—The development of the Leffel wheel is particularly interesting in view of the fact that although originally the two parts were apparently expected to deliver approximately equal quantities of water, the capacity of the upper one has been gradually reduced until in the Samson

* "Mechanics of Machinery and Engineering," Vol. II, p. 254ff.
† *Loc. cit.*, p. 256.
‡ Philadelphia, Pa.
§ Cons. Engr., Ann Arbor, Mich.

wheel, as built in 1905, it embraced less than 10% of the inlet area. At one time, the writer was interested in gaining an idea of the relative effect of the two parts. The upper inlets were closed with blocks, and the wheel was run in that condition until a fairly complete and accurate record of power output had been obtained. The blocks were then removed, in which condition the wheel has been operating for several years. It appears from the record of the plant that, reduced to a uniform head of 17 ft., closing the inlets to the upper section of the wheel reduced the power about 12½% at full gate, about 9% at seven-eighths gate and about 5½% at three-fourths gate. The area of the inlets to the upper section was approximately one-eleventh of the whole inlet area. As no modification of the gates or guides was made, it is apparent that the lower section of the runner would give a greater power if the gates were designed for supplying that part alone, and the increase would be relatively greatest at full gate. Therefore, it seems quite reasonable to state that the upper section at full gate probably contributes its full share of the output, but that its relative influence on the power rapidly decreases with decreasing gate opening.

TABLE 9.—PERFORMANCES OF HOLYOKE TURBINES IN VARIOUS SETTINGS.

Wheels.	Draft chest.	Center bearing.	Distance apart clear.	Horse-power at 16-ft. head, maximum.	Percentage of efficiency.*
2 Original.	Plain	Full	5 ft. 2 in.	268.50	76.02
2 Chipped.	Plain	Small	5 ft. 2 in.	272.50	77.04
2 "	With vertical diagonal plate	None	5 ft. 2 in.	265.75	75.93
2 "	With 2-11-in. interior cones.	Small	5 ft. 6½ in.	276.40	77.60
2 "	" 2-15½-in. " "	"	5 ft. 6½ in.	271.00(?)	76.99(?)
2 "	" 1-15-in. exterior "	"	6 ft. 5 in.	278.00(?)	78.16
2 "	" 1-15-in. " "	"	6 ft. 5 in.	284.00	76.04
2 "	" 2-10-in. exterior and 5-in. interior cones.	Full	6 ft. 10 in.	276.30	78.20
2 "	With 2-10-in. exterior and 5-in. interior cones.	Small	6 ft. 10 in.	286.75	79.43
2 "	With 2-15-in. exterior cones.	"	7 ft. 8 in.	289.75	77.29†
2 "	" 2-15-in. " "	"	7 ft. 8 in.	284.50	81.03
2 New.	" 2-15-in. " "	"	7 ft. 8 in.	289.60	81.38
2 "	" 2-15-in. " "	"	7 ft. 8 in.	289.20	81.60
2 "	" 2-18-in. " "	New	8 ft. 2 in.	288.00	81.50
1 Original.	L. H. vertical cylinder gates	.....	.....	139.95	.....
1 "	R. H. vertical cylinder gates	.....	.....	142.30	.....
	Combined	.....	.....	282.25	.....
1 New.	L. H. vertical wicket gates.	.....	.....	147.10	80.69
1 "	R. H. " " "	.....	.....	149.10	81.14
	Combined	.....	.....	296.20	80.92

* The efficiencies given are based on the corrected discharge of the weir and are between 2 and 3% lower than those reported by the Holyoke testing flume.
† Wheels over-gated.

Michigan Lake Superior Power Company Tests.—From 1899 to 1901, the writer was engaged on an extended series of tests at the Holyoke testing flume for the Michigan Lake Superior Power Company. This was the third series of tests of pairs of horizontal wheels made at Holyoke. The wheels were designed in one of the early attempts to increase the speed of the then recognized type of wheel. As the solution of the problem was then stated, it was getting the power of a 36-in. and the speed of a 30-in. wheel, which was

accomplished with a 33-in. McCormick having a specific speed of 68, whereas that of the usual type was about 50. The wheels were to be used in units of four and were tested in pairs. The requirements were 285 h. p. at 180 rev. per min., with a head of 16 ft. and 80% efficiency for each pair of runners. The wheels were equipped with wicket gates.

The first tests showed the wheels to be seriously handicapped by what the authors have designated as a "boiler-maker" setting. In justice to the wheel designer, the late James F. Jolly of Holyoke, it should be stated that the draft chest was designed without his connivance, and he realized at once what measures should be taken to improve conditions. Numerous expedients were tried, however, and an enumeration of them with the results obtained is presented in Table 9. The first improvement was to enlarge the outlets of the wheels by chipping off the edges of the buckets, thereby increasing the discharge and power. Wheels so treated are indicated as "Chipped" in Tables 9 and 10. Later, a new wheel was designed of larger capacity, which is designated as "New."

The so-called full center bearing originally designed, consisted of a cast-iron girder, 12½ in. deep at the center and 6 in. at the ends, extending across the draft chest, the lower half of the bearing extending 10 in. below it with a diameter of 16¾ in. and a length of 15¾ in. The draft chest was 54 in. in diameter and only 7.0 ft. long with the runners projecting into it about 10 in. at each end, as in the "Boiler Maker Setting 1890" of Fig. 29. The small center bearing consisted of a ring about 2 in. wide and 2 in. deep suspended by three ¾-in. rods attached to the walls of the chest. Although made of cast iron, the original draft chest was a "boiler-maker's setting" with a vengeance, and by the time the tests were concluded it was impossible to find any one who would admit having been responsible for its design. The units were finally equipped with a new chest having conical extensions 18 in. long at each end, and a new center bearing was designed, that occupied about one-fourth the space of the original one. By separating the wheels an additional 6 in., it was possible to get substantially the same results with the new bearing as had been obtained with the small one, with 15-in. extensions (see Table 9), and to more than meet the specifications.

Temperature Effects on Turbine Efficiency.—In the tests of these wheels the effect of cold weather on apparent efficiencies was clearly established, as is shown by Table 10.

In the warm weather tests, the performances of similar pairs of wheels agree within less than 0.5%, both in horse-power and in efficiency, and in the cold weather tests these variations are about twice as great, a difference easily accounted for by the presence of variable quantities of floating ice.

Wheels Nos. 86 and 87 were those tested separately on a vertical setting, with results indicated in Table 9.

A Modern Horizontal Setting.—In 1916, the writer undertook to re-design the hydraulic plant of a large paper mill in Minnesota. The original installation consisted of three pairs of wheels per unit in "boiler-makers" settings, which gave, for a 1-ft. head, 6.3 h. p. at the switchboard, with an efficiency of 48%, equivalent to about 53% for the wheels.

TABLE 10.—Performances of Pairs of Holyoke Turbines, on Horizontal Shaft, Full Gate, 180 Revolutions, 16-Foot Head.

Warm Weather Tests

	Chipped wheels.		New pattern wheels.		
Numbers of wheels	2 and 3	10 and 27	62 and 63		108 and 128
Selected by	Makers	Writer	Makers		Writer
Date of test	Aug. 1900	Aug. 1901	Aug. 1900	Aug. 1901	Aug. 1901
Temperature of water	76° Fahr.	74° Fahr.	76° Fahr.	65° Fahr.	74° Fahr.
Draft case	No. 1*	No. 1*	No 1*	No 1*	No. 1 †
Bearing	Small	New	Small	Small	New
Brake-horse-power	289.60	288.00	292.90	291.00	291.00
Discharge, in cubic feet	196.50	195.10	198.70	198.40	198.50
Efficiency	81.38	81.51	81.40	81.00	80.95

Cold Weather Tests

	New pattern wheels.		New pattern wheels.	
Numbers of wheels	62 and 63		86 and 87	
Selected by	Makers		Writer	
Date of test	Jan. 1901	Jan. 1901	Dec. 1900	
Temperature of water	33° Fahr.	33° Fahr.	37° Fahr.	
Draft case	No. 1*	No. 1*	No. 2*	No. 2*
Bearing	Small	Small	Small	New
Brake-horse-power	288.40	285.50	285.50	283.60
Discharge, in cubic feet	199.40	200.08	199.40	199.30
Efficiency	79.67	78.30	78.85	78.46

* With 15-in. extensions.
† " 18-in. "

These units were replaced by two pairs, as shown on Fig. 42, having the same speed (200 rev. per min.), in settings designed jointly by the writer and the wheel makers, and gave, on test for 1-ft. head, 9.85 h. p. at the switchboard, with an efficiency of 74.6%, equivalent to about 80% for the wheels. In these tests, it was found that by introducing a plate in the draft chest, extending across it and vertically down, the power was decreased to 9.83 h. p. for 1-ft. head and the efficiency at the switchboard was increased to 75.1%, equivalent to about 81% for the wheels. The increase in power of the new setting appears to have been about 56% and of wheel efficiency about 53 per cent. As in these tests the water was measured by current meters, the writer would suggest a possible error of 10% in the test on the original wheels and possibly 5% on those of the new installation.

Scroll Settings.—In February, 1905, the writer designed for the Edison Sault Electric Company of Sault Ste. Marie, Mich., an hydro-electric plant* in which the first low-head, direct-connected, vertical units were installed. The design embraced concrete, open-flume scroll wheel-pits and concrete draft-tubes, with roller bearings supporting the weight of the turbine and the generator, and with conical covers over the turbines. Each unit consisted of a 71-in. vertical Samson turbine making 100 rev. per min., directly connected to a 450-kw. alternator of the umbrella type, designed by the General Electric Company for this plant, on specifications drawn up by Alex Dow, M. Am. Soc.

* *Engineering Record*, November 2d, 1907, p. 483.

C. E. This was the first hydro-electric plant, as far as the writer knows, wherein the electrical equipment was made to conform to the economical hydraulic requirements. In previous plants, the hydraulic machinery was subordinated to the electrical, and the inefficient horizontal installation of multiple-wheel units resulted.

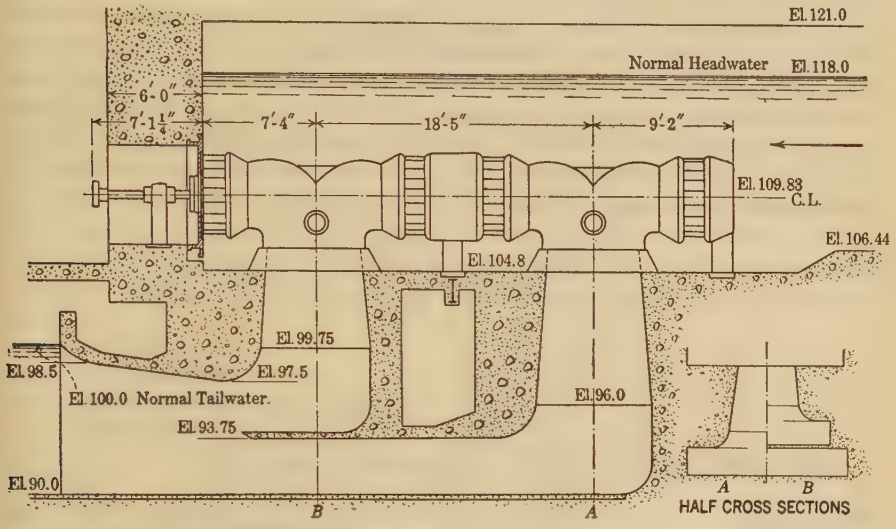

FIG. 42.

Requirement of Constant Speed.—The requirements as to the turbines were complicated by the fact that although the ultimate head to be utilized was expected to be 16 ft., only 12 ft. of this head could be obtained at the start, and in consequence the wheel makers were asked to state what power could be expected from a turbine designed to give 600 h. p. with a 16-ft. head at 100 rev. per min., when it was run at the same speed under a 12-ft. head.

It is an interesting criterion of the condition of the turbine industry in the United States at that time, that only one of those to whom the inquiry was sent, would even guess what power could be expected from the wheel at the lower head, and that one gave 385 h. p. at full gate, but a day later he reduced it to 285 h. p. The correct estimate was about 360 h. p. In July, 1905, the builder, referring to the matter of perfomance under the 12-ft. head at 100 rev. per min., proposed to test at a 12-ft. head and 100 rev. per min. a 56-in. wheel designed for a 16-ft. head and 100 rev. per min. He wrote: "No other wheel has any advantage over us in this matter. We are all in the same condition. We have had no occasion heretofore to make any such test."

Inasmuch as the writer had covered this point quite thoroughly during the tests previously discussed and had prepared a table of "Performances of McCormick Turbines at Over and Under Speeds",* he took the responsibility on himself for the performance at the 12-ft. head and authorized the construction of the wheels. When tested in place, measuring the head from still water

* American Civil Engineers Handbook (4th Edition), p. 1139.

in the forebay to water in the tail-race, and charging against the unit the water used by the exciter, which was independent and water-wheel driven, an efficiency of 68½% was obtained for the power on the switchboard, and correcting for the excitation, the efficiency of the main unit was 71%, which means about 78% wheel efficiency at a 16-ft. head, charging all flume and rack losses against the wheel.

In commenting editorially on other plants, *Engineering Record** stated: "In none of these plants, fortunately, was there need of going to vertical shaft construction of which several examples have recently been seen. The vertical shaft does indeed enable one to keep the generating room above high water mark, but is liable to cause extra cost for attendance on account of step bearings." Nevertheless, ten years later, the standard practice for hydro-electric plant design required the vertical unit.

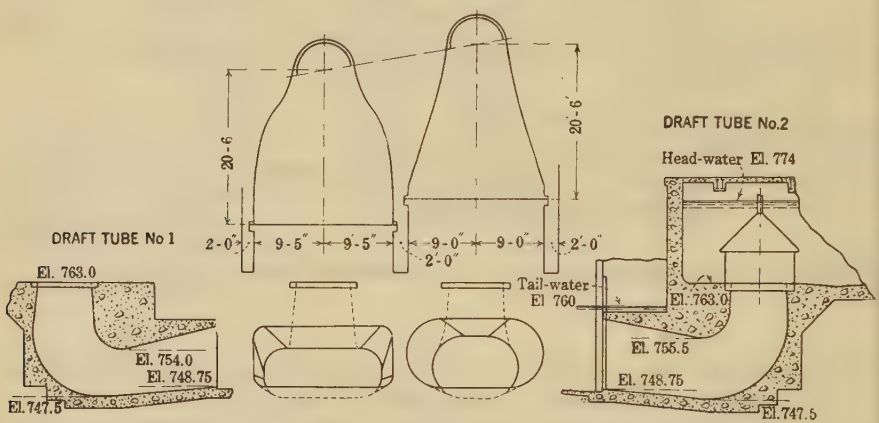

Fig. 43.

Draft-Tubes.—In the plant previously discussed, the concrete expanding draft-tube with a right-angle turn, was introduced, and one of the bidders for the turbines expressed himself as willing to guarantee 1% higher efficiency thereon than for an ordinary setting. The draft-tube was modified in a plant built on the Huron River, near Ypsilanti, and in one on the St. Joseph River for the City of Sturgis, both in Michigan. However, no comparative tests were made, as far as the writer knows, until 1914, when the Argo Plant of the Eastern Michigan Edison Company, at Ann Arbor, Mich., was tested. This plant which had been constructed after the writer's plans and under his supervision, embraced two similar units. The contract for the machinery included a bonus and forfeit clause, and when the writer's design for the draft-tube was submitted, the maker objected to it, declining to guarantee the performance of his machinery on it, and submitted a design of his own which was the recognized standard form for such constructions then and now generally adopted. As in other respects the two settings were practically identical, it was decided to equip one unit with the maker's draft-tube, No. 2, Fig. 43, and one with the writer's design, No. 1, Fig. 43. The plant was equipped with a weir for

* *Engineering Record*, November 2d, 1907.

measuring the water and was tested by Charles M. Allen, M. Am. Soc. C. E., of Worcester, Mass., using an Alden dynamometer. The results obtained are shown in Table 11.

TABLE 11.

	Maximum efficiency, percentage.	Maximum Horse-power.	
		14-ft. head.	1-ft. head.
Maker's draft-tube, No. 2.....	83½	578	11.2
Writer's " " No. 1.....	88	618	11.8

The maker's guaranties were just met on their draft-tube, but he would have won a respectable bonus had he not objected to the writer's design. The maker was not satisfied with the results of the test, and it was repeated when the generators were installed, with similar results, and draft-gauges,

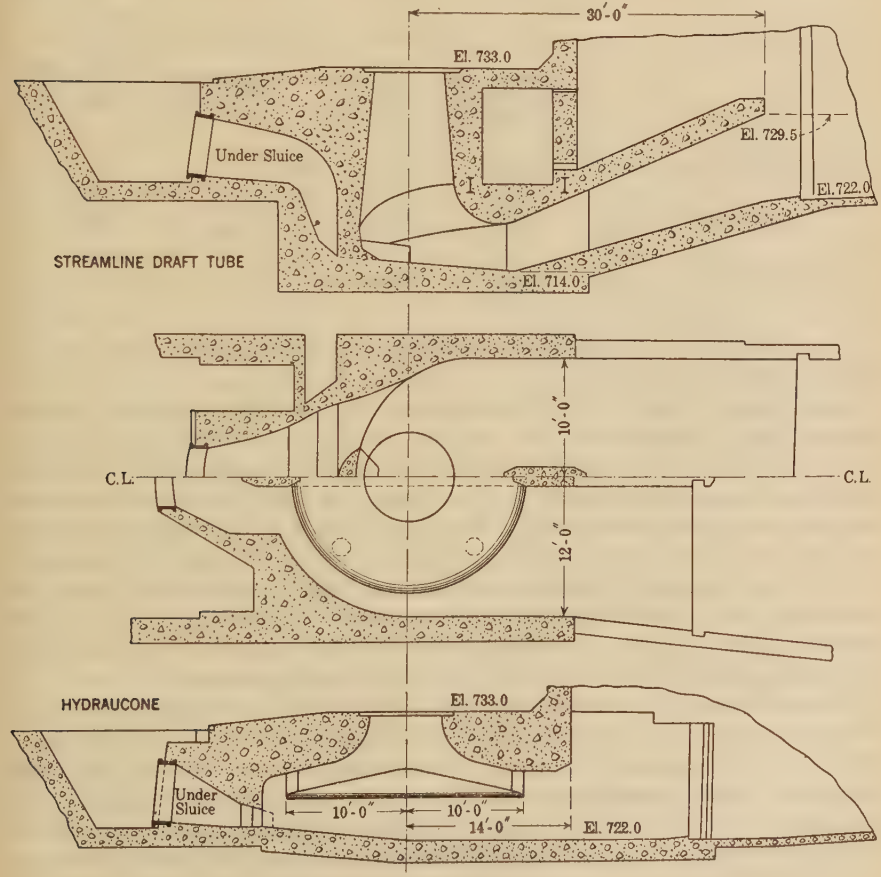

Fig. 44.

attached just below each wheel, showed a considerably higher vacuum for the writer's tube. This removed the last doubt as to the reliability of the result. Meanwhile, a series of tests had been made by the Allis-Chalmers Manufacturing Company of Milwaukee, Wis., on models of draft-tubes, including the Argo draft-tubes, from which it appears that the so called "hydraucone" gave efficiencies between 1 and 2% higher than the writer's Argo draft-tube. The opportunity for further tests was afforded by the construction of the Geddes Plant of the Detroit Edison Company, in which another pair of duplicate units were to be installed. A "hydraucone" was constructed under one unit and a stream-line draft-tube of the writer's design under the other. (Fig. 44.) The wheels were four-bladed Nagler (propeller) runners built by the Allis-Chalmers Manufacturing Company and were identical, as were also all other conditions except the draft-tubes. A weir was constructed across the tail-race for measuring the water. Tests were also made on the "hydraucone" with a three-bladed Nagler runner. The results obtained at 200 rev. per min. are shown in Table 12.

TABLE 12.

	Head, in feet.	Brake-horse-power.	Percentage efficiency.	Corresponding power at 16-ft. head.
Four-blade runner:				
On draft-tube.....	14.11	507.5	83.7	614
	11.87	390	81.2	612
On hydraucone.....	14.59	514.5	81.5	591
	11.77	375.5	78.8	595
Three-blade runner:				
On hydraucone.....	13.26	524.5	80.85	694

The results of these tests on draft-tubes show clearly that the theory on which their design has heretofore been based is largely in error.

Combining the gain of 4½% shown in the Argo test for the writer's draft-tube over the standard type, and the gain of 2% for the stream-line draft-tube at the Geddes Plant over the hydraucone, and the gain of 1% obtained for the hydraucone by its makers over the Argo draft-tube, it appears that the improvements in draft-tubes due to the writer's designs have amounted to at least 7½% in output over the standard design. An increase of 7½% in output means a like increase of income and has been quite a satisfactory return to the owners for the small expense involved in the constructions and tests. If engineers more frequently took advantage of opportunities for comparative tests, technical knowledge would be much more rapidly advanced.

Under-Sluices.—In several of the plants designed by the writer, provision for disposing of low-water flow during construction and for drawing the pond afterward has been made by means of under-sluices connecting with the pond at the inlet of the power house and discharging at the base of the draft-tube. (Fig. 44.) By proportioning the outlet of the under-sluice so as to cause it to discharge the water as a flattened jet along the bottom of the draft-tube, an action somewhat similar to that of an ejector takes place and the flow through

the turbine, and consequently the power, is increased. The device then becomes serviceable to increase the output of the plant at times of high water and, in a measure, to overcome the rise of tail-water. Tests at the Barton Plant of the Detroit Edison Company, have shown an increase of power at the switchboard of as much as 9% of that delivered with the sluice closed at the same head.

Over-Fall as Head Increaser.—Another method of compensating for the loss of head due to high water is that indicated in the Edison Sault Electric Company Plant, wherein the stop-logs above the weir* may be raised by the crane and the surplus water be allowed to fall into the tail-water at the draft-tube outlet and by the resulting depression at the foot of the sheet cause an increase of effective head.

A somewhat similar plan was used for a plant designed to be built on the St. Joseph River in Michigan before the World War. The flood water was to be taken through the power house in channels discharging at the sides of each draft-tube in a nearly horizontal direction, thus obtaining a similarly produced increase of head of greater magnitude than in the more nearly vertical drop at the "Soo".

Accuracy of Holyoke Tests.—The authors quote the statement of the late Robert H. Thurston, M. Am. Soc. C. E., as to the accuracy of the Holyoke tests. With the information Professor Thurston had when he made the statement in 1887, it was justifiable, but such is no longer the case. Investigations made by the writer at the Cornell Hydraulic Laboratory in 1899 brought to light the variations in apparent head that would be indicated by different devices for transmitting the level of the water flowing over the weir to the hook-gauge pail or the gauge-tube. A comparison of the heads indicated by the device in use at Holyoke and that used by Mr. Francis in determining his weir formula, to communicate the head of water to the measuring instrument, has been already presented by the writer.† The original method used at Holyoke for communicating the head in the weir bay to the hook-gauge pail was through a pipe projecting a few inches beyond the wall of the channel, at right angles to the current, the end being cut off square and fully open to the current. The experiments of the late Hiram F. Mills, Hon. M. Am. Soc. C. E.‡ having already shown the erroneous indication of such a device, it was used only temporarily, and was replaced as soon as possible by the transverse perforated pipe now, and since, in use.

If the device used by Mr. Francis gives correct results to be used with his formula and those derived from his experiments, then the Holyoke device does not. For heads on the weir in excess of about 0.8 ft., the error introduced leads to an under-measurement of the water. This causes the reported efficiency to be higher than the true efficiency by amounts varying from zero at 0.8-ft. head to 2.23% at a head of 2.10 ft. on the full length weir, corresponding to variations of discharge between 50 and 205 cu. ft. per sec., as reported in the Holyoke tests. For discharges less than 50 cu. ft. per sec., the Holyoke indica-

* *Engineering Record*, November 2d, 1907, p. 484.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1311.

‡ *Proceedings*, Am. Academy of Arts and Sciences, Vol. VI, New Series, Bost., 1879.

tion of efficiency is slightly low, the error amounting to 0.8% at a discharge of 25 cu. ft. per sec. over the full length weir. The first intimation received by the authorities of the Holyoke flume that the device was in error, was in March, 1900, when the writer made the experimental comparison referred to, and, after a thorough investigation, it was decided that the device should not be changed nor the reports corrected, on account of the large number of tests that had been reported embodying the error. In this decision, the writer concurred at the time, and he still concurs.

The majority of tests made at Holyoke are primarily for determining the relative rather than the absolute merits of the devices tested, and, in the few cases where absolute results are desired, the quantity of water used is generally such that the error is not more than 1 per cent. It is, however, in the writer's opinion, desirable that the facts be known, and in the tests herein reported by him the efficiencies have been corrected to those indicated by the discharge measured according to the Francis method. From his own observations covering many weeks at the flume, the writer feels the utmost confidence in the Holyoke testing flume reports, and in calling attention to the facts, offers no criticism of either its designer or the operators, for it was not until many years after it was put into service that any one appears to have suggested the possibility of a transverse perforated pipe in a flowing stream communicating anything but the true head of water above it.

Floyd A. Nagler,* Assoc. M. Am. Soc. C. E. (by letter).—In assembling the fragmentary records and co-ordinating the steps in the development of the American type of mixed-flow turbine, the authors have made a great contribution to water turbine history. The writer has always been of the opinion that it is erroneous to designate the American mixed-flow turbine as being of the "Francis" type. It might better be called the "Howd" type or equally as well, the Swain or McCormick type, as the last two men are responsible for the most distinctive advancements in the present characteristics of the wheel. As the modern wheel is the product of so many men, all of them Americans, it would be well to drop all personalities in referring to this wheel and call it "the American mixed-flow turbine", as the authors have done.

The paper emphasizes the "scientific designs" of the late James B. Francis, Past-President, Am. Soc. C. E., and gives lesser weight to the advancements made by Messrs. A. M. Swain and John B. McCormick. It is true that Francis seemed to appreciate certain principles of relative velocity and entrance without shock, but he did not correctly apprehend the "centrifugal effect" of water passing through rotating channels† and other principles which render some of his methods of design rather unscientific as compared with those used by the best modern designers. The "cut and try" methods used by Swain and McCormick are not to be regarded as entirely "unscientific", when each new "cut" or "try" is based on previous experience. Much scientific experimentation proceeds along just such lines. Even Francis was guided in

* Asst. Prof. Mechanics and Hydraulics, State Univ. of Iowa, Iowa City, Iowa.
† "Lowell Hydraulic Experiments", p. 42.

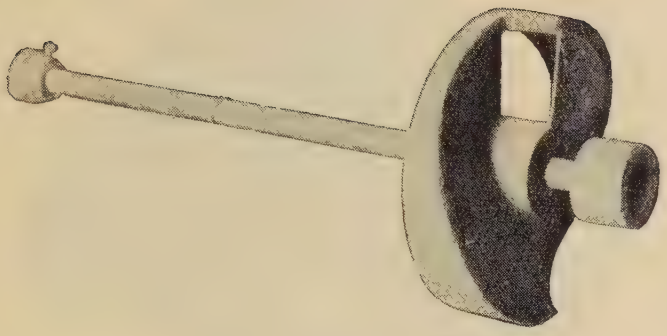

Fig. 45.—First Austin Wheel.

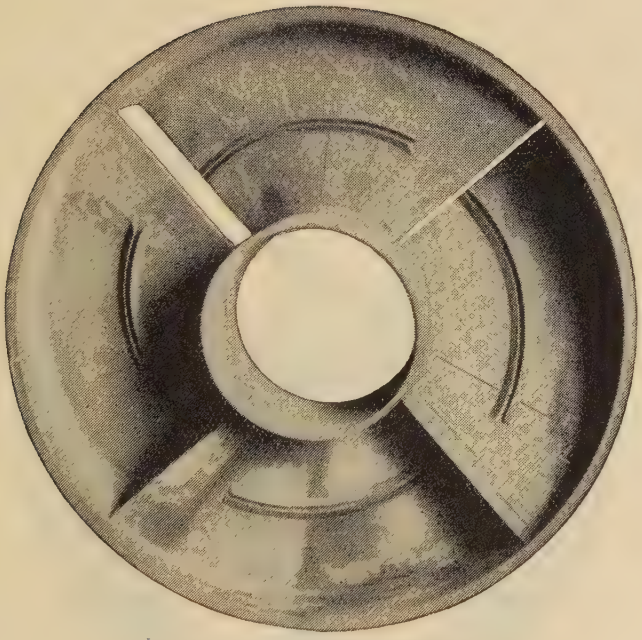

Fig. 46.—Four-Blade Runner of the Austin Type.

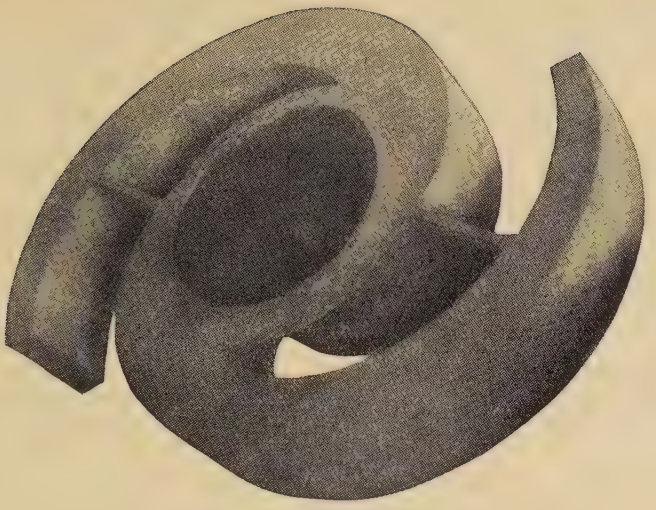

Fig. 47.—Whitelaw Runner of Barker's Mill Type.

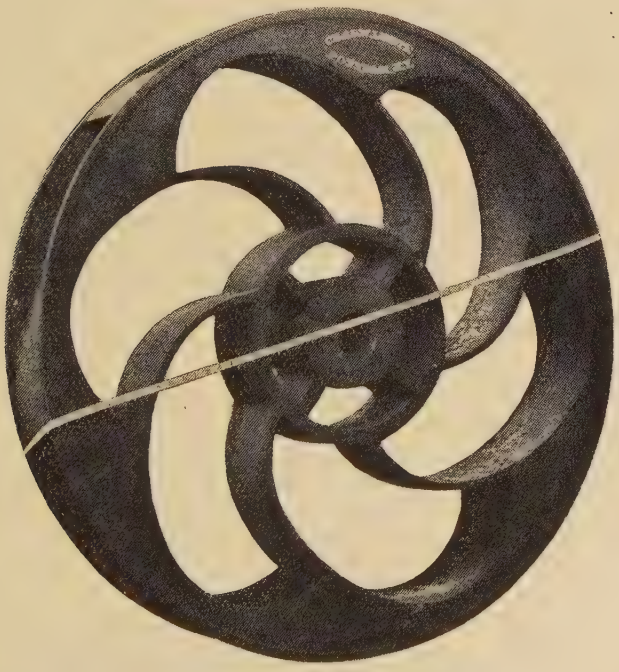

Fig. 48.—The Centrifugal Wheel.

his designs by "but little theoretical information" and "principally by a comparison of the most successful designs"* and developed his rules for proportioning turbines "by the aid of the experiments upon the Tremont turbine".† Too much credit cannot be given McCormick for his contribution in the development of the American mixed-flow turbine, even though his results were accomplished with a great deal of labor and cost. With regard to McCormick's efforts, R. E. Horton, M. Am. Soc. C. E., states:‡

"Probably the greatest achievement of any one man in advancing the development of the hydraulic turbine was that of John B. McCormick of Indiana County, Pa. About 1870, he found that by extending the bucket vanes of an inward flow turbine downward and outward, making them ladle or spoon shaped, he was able to greatly increase the outlet openings of a turbine of a given diameter * * *."

"The form of buckets or water passages through these latest and largest turbine runners differs but little from that developed by John B. McCormick, following earlier attempts at the production of an efficient inward flow turbine runner by Howd, Francis, and Swain * * *."

It would not have been inappropriate for a history of the development of the American type of mixed-flow turbine written in 1922, to have mentioned the name of the most successful designer, Mr. S. J. Zowski (Zwierzchowski), particularly since the authors mention his wheel as having the highest "specific speed" of any of its kind, namely, 102 (pages 1261 and 1269) and illustrate it in Figs. 20 and 21. The fact that Mr. Zowski is of foreign birth and that he applied his theories, which were learned in European schools, to the perfection of the American type of turbine is not sufficient reason for the failure to mention his name in the paper. Mr. Zowski is an American citizen and has been a thorough student of the American type of water turbine§ as he found it when he came to the United States, but he went further.

The authors closely follow the development of the turbine until maximum values for specific speeds of 68 and 69 were attained in 1899 and 1897 by the Smith-McCormick and Samson runners, respectively (Table 1); then it is stated that a present-day model has a "specific speed" of 102. Certainly, an increase in the specific speed of a runner from 69 to 102 indicates that some radical changes must have taken place in its structure, and represents a greater betterment of the turbine speed and capacity characteristics than those obtained by Francis, Swain, or McCormick. There can be little doubt but that Mr. Zowski was the leader in this latter achievement, and his wheels to-day represent the runners of highest efficiency and "type characteristic" manufactured by three of the largest producers of the "American" type of water turbine. The Zowski models numbered, I, II, III, IV, V, and VI, had "specific speeds" of 87.4, 92.8, 78.0, 90.0, 91.0, and 102.0, respectively, with

* "Lowell Hydraulic Experiments", p. 7.

† *Loc. cit.*, p. 44.

‡ The Encyclopedia Americana, "Water Power", p. 29, 1920 Edition, and *Engineering News-Record*, October 7th, 1920, p. 685.

§ "The American High Speed Runners for Water Turbines", S. J. Zowski, *Michigan Technic*, June, 1908; "A Rational Method of Determining the Principal Dimensions of Water-Turbine Runners", S. J. Zowski. *Michigan Technic*, 1909; "Some Recent Tests of High-Power, High-Speed Water Turbines", *Engineering Record*, November 28th, and December 26th, 1914.

maximum efficiencies of 90.0, 87.2, 83.2, 89.2, 89.3, 90.1, and 90.7, respectively. In addition, his runners showed a flexibility hitherto unrealized.*

On page 1262, it is stated that the makers of the "Wynkoop" wheel claimed an efficiency of 175% for their runner. Mr. Emerson states† that an efficiency of only 135% was claimed, based on figures quoted from the maker's table. Exaggerated as this last figure may be, it is a more accurate statement of the maker's contentions.

Several years ago, the writer had occasion to study the history and characteristics of the Austin wheel, mentioned on page 1265. The first wheel produced by Mr. Austin, shown in Fig. 45, was only 14.5 in. in diameter, and was patented by him in 1878. This runner, together with another 4.0 ft. in diameter, are the property of Mr. C. E. Kinne, of Watertown, N. Y. It was reported to the writer by Mr. Kinne that, on test, this 4.0-ft. Austin runner, "under 9-ft. head, made 150 rev. per min. under no load, giving a rim speed 27% faster than the spouting velocity of the water driving it; when loaded, it ran at 125 rev. per min., with a rim speed 6% faster than the spouting velocity of the water". Based on these figures and on measurements of the openings through the runner, and using the most favorable coefficients in computation, the writer has never been able to ascribe a specific speed greater than 60 to this runner. This type of wheel does not have a large capacity for water, even though its speed is very high.

In the attic of the Leete Iron Foundry, at Potsdam, N. Y., the writer photographed patterns of what appeared to be a four-blade wheel of this same type shown in Fig. 46. A runner made from this pattern, which was 4.0 ft. in diameter, should have more than twice the capacity of the 4.0-ft. Austin wheel, but no doubt would have less speed. If the Truax Green Mountain wheel performed in a similar manner, the specific speed of 125 ascribed to it in Table 1, based on claims made by the makers, is far too high.

A Whitelaw runner of the Barker's Mill type, similar to that shown in Fig. 4, is given in Fig. 47, and was photographed by the writer in the collection of old water-wheels of the Bagley and Sewall Company, at Watertown, N. Y.

Fig. 48 shows a runner which is an excellent example of the "cut and try" period. This wheel, commonly called the "Centrifugal Wheel" was first manufactured in the C. W. Leete Iron Foundry, at Potsdam, N. Y., in 1870. The wheel was photographed in the scrap heap of the Leete Foundry in 1919.

Harvey Linton,‡ M. Am. Soc. C. E. (by letter).—Believing it to be sufficiently worthy of notice to be given a place in the history of water-wheel design and practice in the United States, so ably presented by the authors, the writer submits, from notes made at the time, the performance of a water-wheel of the Seventies.

The water from the "Spring", 788 cu. ft. per min., flowed quietly away. A few inches under the surface was a 31-in. wooden turbine, making

* *Engineering Record*, November 28th and December 26th, 1914.

† "Emerson's Hydraulics, Dynamics, etc", 6th Edition, p. 193.

‡ Philadelphia, Pa.

157 rev. per min., that furnished the power to drive a 4-ft. mill-stone and the necessary elevators, bolting machinery, etc.

Except for the grist-mill and penstock, this noiseless little water-wheel, which had displaced an iron turbine, might easily have escaped notice. It worked under a head of 7 ft., and was estimated to develop 8⅓ h. p. That was in 1877.

In 1880, the owner's testimony was as follows:

"It continues to give the best satisfaction, even seems to work better than it did at first, * * * and to be as sound and good as when new. It does more work than was expected of it. The 36-in. Franklin turbine in the same penstock, will not compare with it in power and economy of water."

The miller stated that:

"It grinds more than 5 bushels of wheat per hour, running one smutter, three bolts, elevators, one middlings purifier, and other machinery. It has fully 50% more power than the Franklin wheel and gives no trouble by clogging with drift, which is a serious objection to the iron wheel."

The appearance of that "Spring" indicated that all the force of the 7-ft. head of water was expended in useful work. This result was effected without the use of the numerous small openings and guides usually seen in a turbine wheel-case.

At another grist-mill, a few miles distant from the pioneer installation, an 11-in. cast-iron turbine, constructed on the same general plan as the wheel previously described, displaced a 17½-ft. over-shot wheel. This little turbine was rated at 5 h. p., under a 20-ft. head of water.

In 1880 the owner said of it:

"It does one-third more work in the same time than the 17½-ft. over-shot wheel which it displaced, using about the same quantity of water. * * * My old over-shot was frequently frozen up a great part of every winter."

The testimony of all users of this form of turbine was of similar character. The inventor was Robert Wilson, of McLeansville, N. C. (Patent No. 168 202 dated September 28th, 1875.) He found such demand for his turbine (named by him, "The Inclined Plane Water-Wheel") that, with the assistance of a number of millwrights, he abandoned his milling business and gave all his time to making and installing his wooden turbines. His methods do not appear to have been of the "cut-and-try" character, but were based on something more like close observation and a regard for the principles of hydraulics, that always brought good results.

The apparently better class of turbines (constructed of cast iron), in use at that time, were more extensively advertised than the Wilson wheel. They were often failures; as much, perhaps, because they were not suited to the work put on them, nor to the sites for which they were chosen, as on account of their being inferior to this wooden wheel. The Wilson turbine was always built for the particular site and the work required of it. Its working speed, in revolutions per minute, was found to be the theoretical velocity of water, in feet per minute, discharged under the actual working head, divided by the

circumference of the wheel, in feet. This working speed was said to be higher than that of any other turbine of its time.

Figs. 49 to 53, inclusive, show the construction of a 28-in., upward discharge, wooden turbine and "chest", as prescribed by Robert Wilson. Fig. 49 is the plan of the wheel; Fig. 50 shows the interior of the wheel with half its staves removed, the "head block", and the depth of the "chest"; Fig. 51 shows the plane projection of the outer ends of the "inclines," having a radius of 14⅜ in.; Fig. 52 is a horizontal section of the chest and penstock; and Fig. 53 is a side view of the wheel, chest, and penstock. The level of the tail-water is a little above the level of the top of the wheel when at work. Details, such as cast-iron flanges for the wheel shaft, bearings, etc., are not shown.

Robert Wilson's rules for constructing his turbine were, briefly, as follows: Knowing the flow of the stream, in cubic feet per minute, and the head of water over the turbine at work, he assumed that $A = \frac{Q}{\frac{4}{9}\,q}$, in which A is the area, in square inches, of the sum of the areas of the openings in his turbine that will discharge a volume of water yielding 1 actual h. p., assuming its efficiency to be 80%; Q is the volume of water, in cubic feet per minute, discharged by the turbine to develop 1 h. p.; and q is the theoretical discharge, in cubic feet per minute, of water under the given head, through an aperture 1 in. square.

In this 28-in. wheel, there are four orifices, each 6 by 7 in., totaling 168 sq. in.; the head is 8½ ft.; and A is found to be 17.9026 sq. in.; then,

$$\frac{168}{17.9026} = 9.384 \text{ h. p.}$$

The stream flow being 728.634 cu. ft. per min.,

$$\frac{728.634}{77.647}, \text{ or } \frac{\text{stream flow}}{Q} = 9.384 \text{ h. p.}$$

$Q = 77.647$ cu. ft. per min.; $q = 9.7587$ cu. ft. per min.

$$A = \frac{Q}{\frac{4}{9}\,q} = \frac{77.647}{4.3372} = 17.9026 \text{ sq. in.}$$

The working speed is:

$$\frac{1\,405.249}{7.3304} = 191.5 \text{ rev. per min.}$$

The "draft" for wooden turbines is 1½, that is, 1½ in. measured on the circumference of the hub, to a 1-in. drop of the "inclines". For iron turbines, the draft recommended is 1 to 1.

The inventor claimed 80% efficiency for his wheel, judging entirely by results: he had no faith in the Emerson tests. In one instance, a 52-in. downward-discharge wooden turbine, with four orifices in the chest, displaced an iron turbine of the same size, taking its place on the same shaft, to drive a 52-in. circular saw. A draft-tube was used in another location.

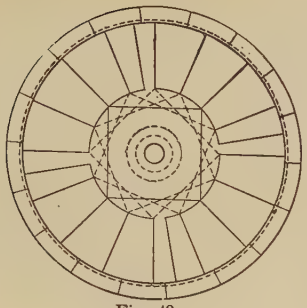

Fig. 49

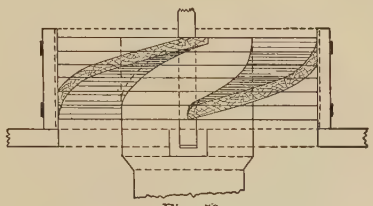

Fig. 50

Fig. 51

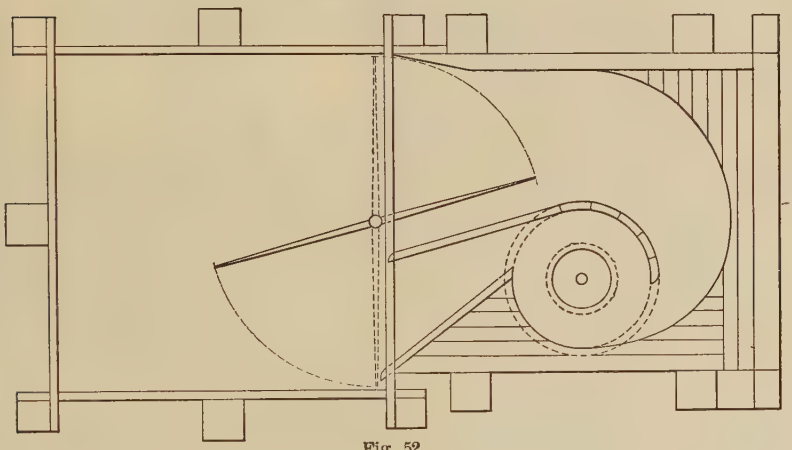

Fig. 52

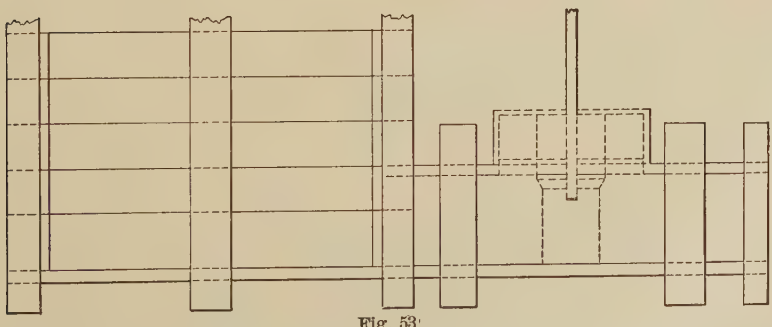

Fig. 53

These wooden wheels could easily be rebuilt if the millwright had the original plans, and they could be used to advantage by many owners of small water-powers to-day. They were always constructed of green timber, yellow pine being preferred. Reinforced concrete could be used to advantage in the construction of the wheel-chest.

ARTHUR T. SAFFORD,* M. AM. SOC. C. E., and EDWARD PIERCE HAMILTON,† ESQ. (by letter).—A somewhat more extended, although still far from complete, study of the early American water-wheels disclosed several interesting examples. Mr. C. E. Kinne, of Watertown, N. Y., who is much interested in water-wheel history, has been of great assistance to the writers. It was from him that the photographs of the early reaction wheel, the Scotch Mill, Austin, and Truax wheels were obtained.

Isaac Sanderson, of Watertown, Mass., built a paper mill at the lower falls of the Neponset River, at Milton, Mass., in 1817, and installed a wrought-iron tub wheel.‡ This is the earliest record the writers have found of an iron turbine. It is especially interesting that this wheel should have been installed at the place where water power was first developed in the United States. The lower Neponset Valley was one of the earliest industrial centers of the country, and a variety of manufactures were in operation there previous to 1700.

The 1840 reaction wheel (Fig. 54) was set at the bottom of an open flume without any guides, and discharged outward. Wheels of this model were common in Oneida County, New York, during the middle of the Nineteenth Century. The "Scotch Mill", a modification of Barker's Mill, was invented by Whitelaw and Starrett, of Paisley, Scotland, and patented in the United States in 1843. It is said that millers whose mills contained these wheels often had to suspend operations and devote an afternoon to chiseling a muskrat out of one of the tapering arms. The example shown (Fig. 55), was running in a grist-mill as late as May, 1919. The Austin wheel is mentioned on page 1265. Fig. 56 shows what is claimed to be the first Austin wheel made.

Probably the first of the reaction wheels to come into use in New England was that of Calvin Wing, of Maine, which appears to have been invented in 1833. It was somewhat like a Boyden runner, consisting of top and bottom circular plates with vanes extending between them. The upper plate was pierced by a number of holes through which the water was fed to the interior of the wheel, whence it discharged outward. These holes were of less area than the bucket orifices and, consequently, the flow was restricted and the efficiency poor. Some of these wheels were installed at the Boston and Roxbury Mill Dam about 1835.

Mr. Forrest Nagler feels that too much credit cannot be given to Swain for his mixed-flow turbine. A search through the personal and technical papers of U. A. Boyden has revealed the following facts regarding the invention of the mixed-flow turbine. In May, 1860, Swain was granted a patent on

* Engr., Proprietors of Locks and Canals; Cons. Hydr. Engr., Lowell, Mass.
† Milton, Mass.
‡ Teele's "History of Milton", p. 372, Boston, 1887.

Fig. 54.—Reaction Wheel, 1835.

Fig. 55.—Scotch Mill, 1843.

FIG. 56.—AUSTIN WHEEL.

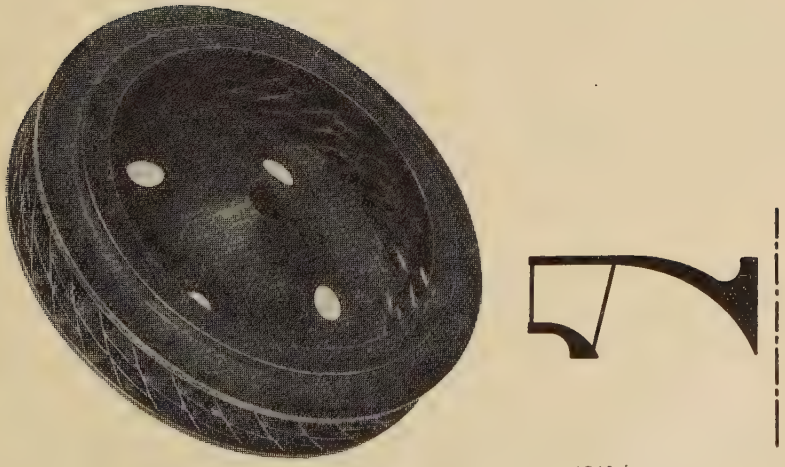

FIG. 57.—BOYDEN'S MIXED-FLOW RUNNER, 1849.

a water-wheel covering three claims for the arrangement of the cylinder gate and the adjustable step-bearing. Nothing was claimed for the runner. In November, 1872, a substitute patent was issued with several additional claims, among which was one covering an inward-flow water-wheel with the discharge edge extending from the crown to the bottom edge of the band. This claim was broad enough to cover almost every mixed-flow wheel made. The Swain Turbine and Manufacturing Company tried to restrain James Ladd from infringing this re-issued patent. The bill was dismissed by the United States Circuit Court for the Massachusetts District, in January, 1877, and on appeal to the Supreme Court, this decision was upheld. The truth appears to be that Swain did not even attempt to patent the mixed-flow turbine at the time of the invention of his first wheel. He assigned the re-issued patent, which was extended to cover such a wheel, to the Swain Turbine and Manufacturing Company, and this Company attempted to secure a monopoly on all mixed-flow turbines.

Justice Bradley delivered the opinion of the Supreme Court (102 U. S. 408). After covering the legal points of the case, he stated that the Stowe wheel, built in 1837, 1841, and 1850, although an impulse wheel, had buckets of essentially the same shape as Swain's wheel. He also cited three wheels, each of which he said was the same as Swain's and that if any of the three was of a date previous to 1860, it clearly showed that the idea of the mixed-flow principle was not new. These wheels were the Temple (1859), the Whitney (1864), and the Greenleaf (1854).

Swain tried to extend his first patent by a re-issue to cover a principle which he may or may not have invented, but which he did not claim in his first patent, and this principle had come into common use since that time. It was held that a re-issued patent could not be extended to cover claims not in the first patent and that, although Swain may have been the inventor of the mixed-flow principle, he could not at this late period secure a monopoly of it.

On June 13th, 1849, Uriah Boyden applied for a patent on an inward-flow turbine. Fig. 57 shows a photograph of one of his original models now in the possession of John R. Freeman, President, Am. Soc. C. E. Notice the horizontal dividing diaphragms which were to appear later as the fins on the "Hercules". They were designed to lessen the part-gate contraction of the cylinder gate. The following part of a letter tells the story in Boyden's own words:

"Boston, July 26, in 1876.

"To the Judges of the Circuit Court
of The United States for the District of Massachusetts:

"I do not presume to know whether any general rule against noticing informal writing should prevent you from noticing this letter. I will deliver a duplicate of it to the complainant or its counsel, and I will also deliver a duplicate of it to the defendant or to his counsel.

"Data indicate that the suit of the Swain Turbine and Manufacturing Company against James E. Ladd in the Circuit Court of The United States for Massachusetts includes sham which you do not know. I tried to prevent anything from connecting me with said Company and their associates manufacturing water-wheels; and I do not intend to include in this letter anything which causes do not urge me to mention.

"Hydraulics motors partly like Asa M. Swain's turbines have been called Poncelet turbines, Howd turbines, inward flow turbines, center vent turbines, and converging turbines. I applied for a patent for improvements chiefly in said kind of motors; and some years after I applied, the United States issued a patent to me for them, and numbered it 10,027 and dated it September 20 in 1853; and the next two quotations are paragraphs from it.

" 'In wheels of this kind, with which the water is directed obliquely to the wheels by the guides * * *, or by other things, the velocity of the water diminishes, or should, from the time it strikes the floats till it leaves them or till near the time it leaves them; and the change of the directions of its motion which should be produced during the same time is not sufficient to compensate the effects of the diminution of its velocity and the less space the water has in consequence of its approaching the axes of the wheels, so that the passages between the parts of the floats nearest the axes of the wheels are choked some. To produce the greatest possible effect of the water with this kind of motors, it is necessary that the resistance to the moving of the water through the wheels should be less than it is in them as usually made, so that the velocity of the water on entering these wheels should be greater than it usually is. There is no such disadvantage of partial choking in mere re-acting wheels, in which the velocity of the water increases as it passes through the wheels, and in which much contraction of the passages is needful. And this partial choking is also different from anything which happens in the common or Fourneyron turbines, in which the water diverges as it passes between the floats and out of the wheels at their peripheries instead of converging as in the Poncelet turbines. This disadvantage cannot be obviated by placing the inner parts of the floats farther apart, without their being directed so nearly toward the axis of the wheel as to cause a part of the force of the water to be lost. The fourth branch of my invention consists in making the distances between the rims of the wheels at the ends of the floats next the axes of the wheels greater than at their outer ends, * * *. This difference in distances may be made by making so much of the upper part of the wheel as the upper edges of the floats touch plane or flat; and the upper surface of the lower rim next its periphery, including about one-eighth of the width of this rim, also plane; and from this place gradually curving downward, as by an arc of a circle; the curvature of this part being such that it will join the plane part without any sensible angle at the place of joining, and the distance of that part of the lower rim at the inner ends of the floats from the upper rim will be about 25% greater than at their peripheries. It is well to have the upper edges of the floats rather longer than their lower edges, so that the tops of their inner ends will be rather nearer the axes of the wheels than the bottoms of their inner ends; * * *.' "

The next following paragraph includes the claim of the improvement above described.

" 'Fourthly, the shape of the spaces between the rims of water-wheels which the floats are fastened to, in which they flare toward the axes of the wheels; as above described; though I do not limit my claim to exactly the placing above described, but extend it to all placing which will essentially answer the same purpose. The first, third, and fourth branches of my claim apply only to such hydraulic motors as have guides or other things which cause the water to move obliquely toward the wheels in the way in which the wheels turn, and pass into the wheels at their circumferential parts, and after acting on the floats discharge from the floats inward. I do not extend these divisions of my claim to the class of tub wheels and undershot wheels in which the water generally flows into the wheels in streams with spaces between the streams, at which spaces the water does not flow into the wheels. Though I have described these water-wheels as being horizontal and the gates as being opened by raising, it is obvious that all these four branches of my claim are quite applicable to

wheels in other positions, and to cases in which the gate is opened by lowering, and I do not limit either branch of my claim to cases in which the wheels are horizontal or to cases in which the gates are opened by raising.'

"Gay, Silver and Company made excellent turbines according to my design and under my supervision, and subsequently, in A. D. 1854, I made an elaborate design for a converging turbine with such flaring of the spaces between its rims as is claimed in said patent, for actuating their machinery at North Chelmsford in Massachusetts, and they made a turbine according to that design exclusively of my supervision, which they did not need, and actuated their machinery. [This is the wheel shown in Fig. 24. It ran for more than fifty years before it was taken out about fifteen years ago.] I never removed that design from their shop. Harvey Silver of said firm, and other men, told me that said firm made spaces between its floats flare toward its axis and data indicate that he saw a copy of my said patent, but page 215 of the printed evidence shows that he testified or implied that the spaces between the floats of my converging turbines did not flare toward the axes of their wheels, so as by their flaring to produce any substantial effect on water's motions in them; which is an error.

"When such a converging turbine as I claimed works with its gate fully open, the water which flows through the upper parts of the spaces between its floats flows toward or nearly toward its axis when it leaves them, and it then moves horizontally or nearly so; and the water which flows through the lower parts of the spaces inclines downward when it leaves them, so that the inclinations downward at different distances below the tops of the floats are somewhat proportional to these distances.

"Asa M. Swain's patent dated May 15 in 1860, for alleged improvements in converging turbines, and the substitute for it dated November 19 in 1872, represent a cylindrical piece around the lower parts of the floats, which has its diameter as large or nigh as large as the upper rim or part which joins the upper edges of the floats, and this piece is partly a substitute for the lower rim of a common converging turbine, so that it partly answers one of the purposes of such a rim. The upper outer part of each of his floats is nearly as described in my said patent, and from their upper parts his floats extend downward by double curvature to said cylindrical piece, so that the length of the discharging orifice between any two floats is greater than the height of the receiving orifice of the space between the same two floats; and it is greater than I showed in said patent, and his form involves wasting more of the power of water which actuates a turbine than the form which I showed by my said patent. The printed bill of complaint with the printed evidence represents that this suit is to check or stop alleged infringing a patent right to this one of Swain's alleged improvements.

"Waters implied that he did not know or believe that previous to Swain's alleged inventing, any converging turbine was known in which the spaces between its floats had their discharging orifices longer than the height of their receiving orifices, which I had claimed in said patent; and much of his testimony in the printed evidence is on some advantage of such excess of length, and he testified or implied that because Swain's turbines had such excess they were better than other turbines; yet the printed evidence shows that Waters and other witnesses neglected or blinked elements in deducing that inference and centrifugal force is one of those elements, and I next quote, for writing on some of its effects. * * *."

This wheel of 1849 was a mixed-flow turbine. Let us now find the source from which Boyden derived this idea, for it seems that it did not originate with him. Among the papers of the late James B. Francis, there is a memorandum, a part of which is quoted herewith:

"* * * I met Mr. Boyden on the street. I asked him if he did not think that the making the distance between the crowns of the wheel greater on the inside periphery than on the outside was new, also the mode of placing the gate between the guides and the wheel. He replied that he had application then pending for patents for those two things. I was much surprised—the more so as I had explained to him some months ago that I intended to make the wheels for the Boott Mill with a greater height between the crowns on the inside periphery than on the outside—and the impression I got at the time was that he thought it would not answer and he gave me not the slightest intimation that he claimed it as his invention. * * *

"I do not know the date of Mr. Boyden's application to the patent office, but I know that the improvements as applied to center vent wheels above mentioned were made by myself and that I did not derive them or any hint of them as applied to this kind of wheel from Mr. Boyden or any one else."

"*Signed:* JAMES B. FRANCIS."

"LOWELL, April 15, 1848.

The model Lakin wheel shown in Fig. 25 was tested on February 16th, 1922. The power was measured on the jack-shaft by a small Prony brake, the water by a calibrated 3-in. meter. A maximum efficiency of 58.7% was obtained, despite the fact that the runner is only 4½ in. in diameter. It was also found that, at the lower speeds, the governor functioned smoothly and held the wheel to its speed.

Fig. 58 shows a Curtis wheel, which is described on page 1282. A 32-in. Truax runner (Fig. 59), kindly lent by Mr. Kinne, was tested on July 6th, 7th and 8th, 1922, in the flume of the Proprietors of the Locks and Canals. The runner was first fitted with a wooden draft-tube 5 ft. long and a discharge diameter of 4 ft. A wooden casing, built according to the directions furnished by the maker for this size of wheel, was constructed around the runner for the first test. The runner was then tested with the casing, but without the draft-tube, and, finally, without either. Table 13 and Fig. 60 show the results of these tests. The results clearly prove the contention of the writers that a high-speed propeller wheel was on the market fifty years ago. Moreover, the characteristic curve of the Truax wheel closely resembles that of the modern propeller type (Fig. 61), and, although low in efficiency, has nearly the same speed.

The results of this test are very different from those obtained by Mr. Forrest Nagler. The conditions, except for the addition of the draft-tube, were identical with those advised by the maker, and the use of the draft-tube on this particular wheel had been suggested. (See page 1286.)

Mr. Nagler's conditions were not fair to the wheel. The casing of the wheel is an essential part of the whole, particularly under a comparatively high head, such as Mr. Nagler used. Under very low heads, the omission of the casing is not so vital to the speed, as a vortex forms, which partly takes its place. Under higher heads, there would be no vortex, and the result would be the same as putting the wheel in a jet of water which had no whirling component. This is clearly shown by the very low speed obtained with the 10-in. model. Mr. Nagler's model under 13-ft. head ran on the same principle as Barker's Mill and not as a propeller wheel, to the successful operation of which

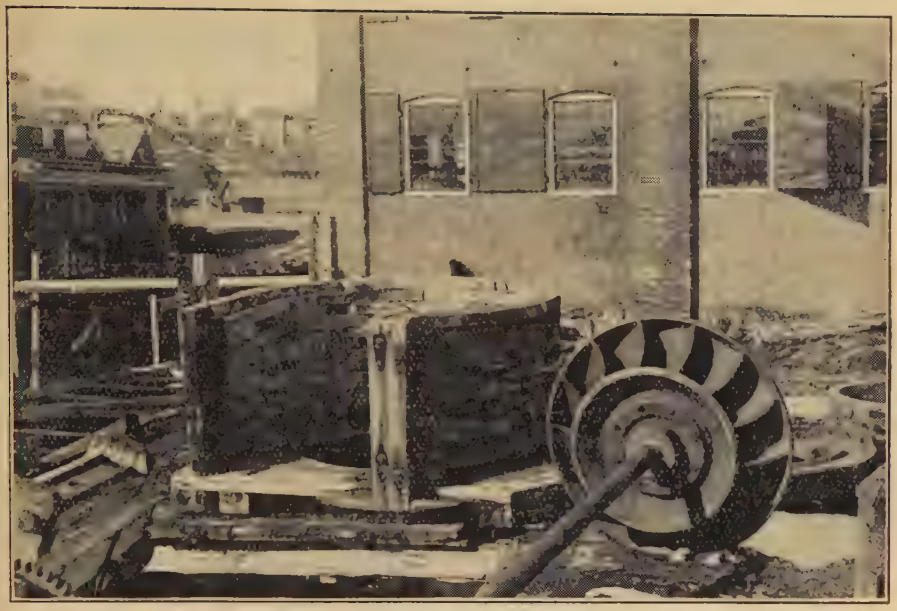

FIG. 58.—CURTIS WHEEL.

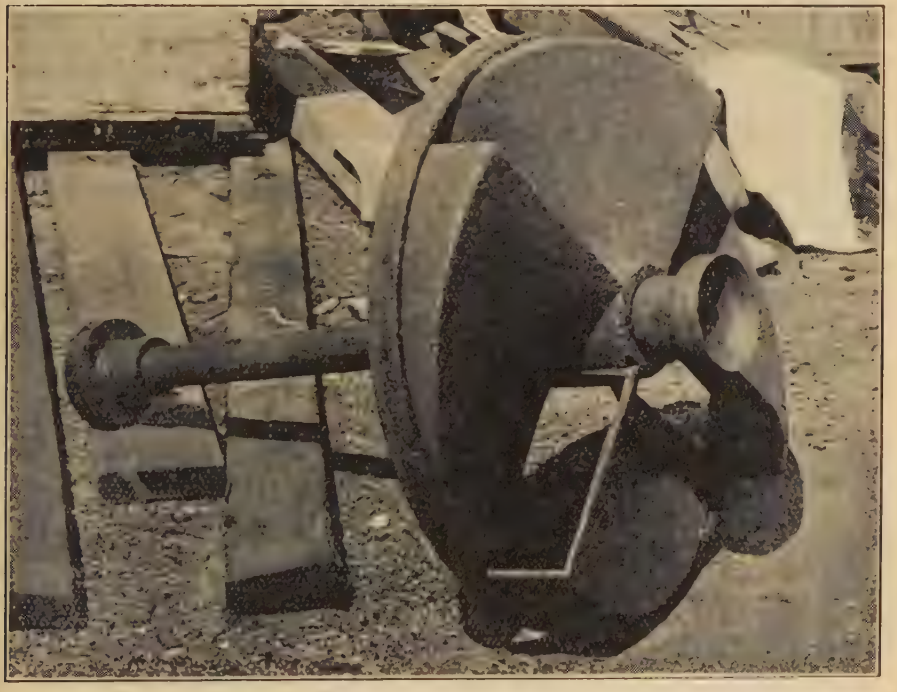

FIG. 59.—TRUAX RUNNER.

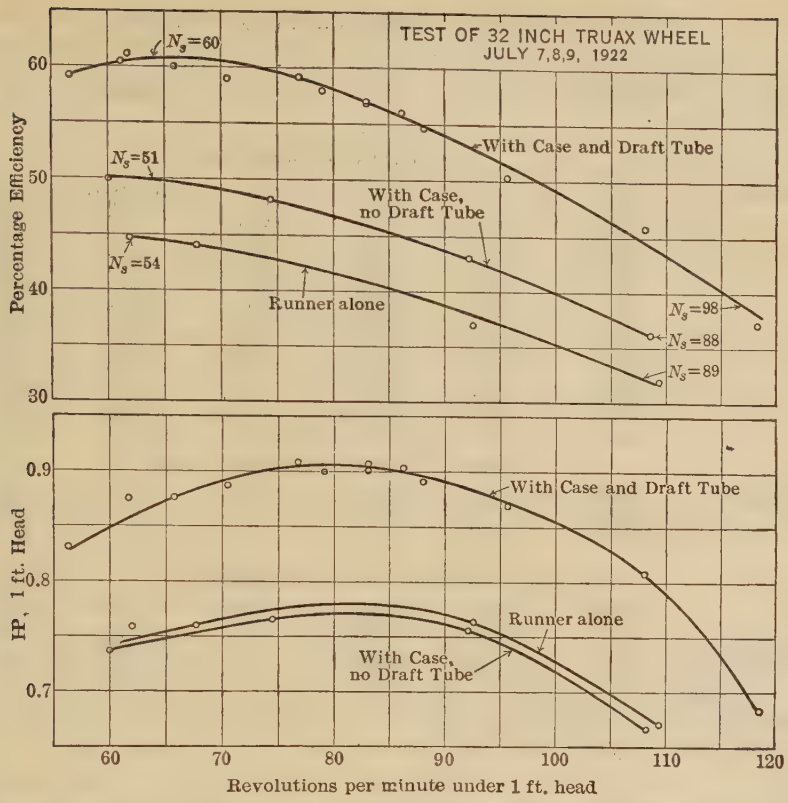
TEST OF 32 INCH TRUAX WHEEL
JULY 7,8,9, 1922
Percentage Efficiency
N_s=60
N_s=51
N_s=54
With Case and Draft Tube
With Case, no Draft Tube
Runner alone
N_s=98
N_s=88
N_s=89
HP, 1 ft. Head
With Case and Draft Tube
Runner alone
With Case, no Draft Tube
Revolutions per minute under 1 ft. head

FIG. 60.

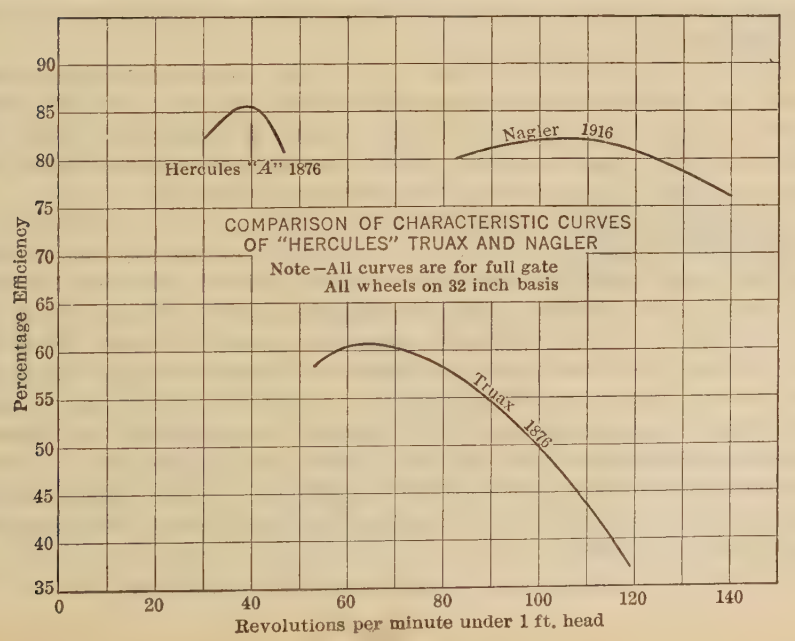
Percentage Efficiency
Hercules "A" 1876
Nagler 1916
COMPARISON OF CHARACTERISTIC CURVES
OF "HERCULES" TRUAX AND NAGLER
Note—All curves are for full gate
All wheels on 32 inch basis
Truax 1876
Revolutions per minute under 1 ft. head

FIG. 61.

TABLE 13.—TESTS ON 32-INCH TRUAX WHEEL.

Date.	Revolutions per minute.	Head.	Q.	Horse-power.	Efficiency.	N_s
			WITH CASING AND DRAFT-TUBE			
July 6, 1922*...	333	5.50	.....	.....	Runaway	....
	198.5	5.71	33.35	12.30	57.0	78.7
	197.7	5.68	33.31	12.24	57.2	79.1
	204.7	5.66	33.66	12.18	56.2	81.8
	209.3	5.64	33.98	11.92	54.8	83.0
	182.7	5.66	32.14	12.22	59.3	73.1
	188.3	5.67	32.46	12.14	58.1	75.0
	168.5	5.71	31.68	12.12	59.2	66.4
	158.0	5.79	30.88	12.16	60.0	61.4
	147.7	5.85	30.26	12.10	60.3	56.4
	136.7	5.89	30.00	11.88	59.3	51.4
	228.7	5.78	36.34	11.90	50.4	89.0
	255.8	5.61	36.82	10.75	45.9	97.1
	278.3	5.52	38.34	8.91	37.1	98.0
	294.5	5.51	39.68	6.48	26.2	88.9
	148.3	5.78	30.34	12.16	61.2	57.6
			WITH CASING, BUT WITHOUT DRAFT-TUBE			
July 7, 1922†...	132.3	4.88	28.66	7.94	50.1	51.4
	164.3	4.88	30.82	8.22	48.1	65.1
	202.7	4.87	34.14	8.11	43.0	80.0
	239.5	4.89	35.90	7.18	36.1	88.4
			NO CASING AND NO DRAFT-TUBE			
	203.5	4.85	39.87	8.14	37.1	80.8
	242.5	4.91	41.00	7.28	31.9	89.6
	348	4.95	.....	.....	runaway	....
July 8, 1922†...	150	4.91	33.59	8.25	44.1	58.9
	137.3	4.91	33.13	8.24	44.7	54.0

* Temperature of water = 66.5° Fahr.
† Temperature of water = 67° Fahr.

a whirling component in the water is essential. A modern Nagler runner would not have given a high-speed performance under the conditions imposed on the Truax by Mr. Nagler.

The writers do not wish to detract from the very valuable development work done by Mr. Forrest Nagler in recent years on the propeller wheel. They feel, however, that the propeller type of wheel is not a new development, but has existed for many years in a state which, although crude and inefficient, had the same general characteristics as the present-day models.

It must be remembered that the tests of fifty years ago, which showed efficiencies of 90%, are not absolutely substantiated. Emerson was a self-trained man with a great scorn for technical training of any kind other than "cut and try". All his tests previous to the establishment of the Holyoke Flume are doubtful, and the results must be considered as comparative only. What Mr. Scheidenhelm says about these high efficiencies being reached by exceptional wheels is well taken. Emerson says of the Risdon wheel, "* * * quite a number of them have ranged along in the seventies in percentage, but, through some slight change after a first trial, every wheel tested (except two or three of the 20-in. size), has been made to return a useful effect of over eighty per cent. before delivery to purchaser, quite a number from eighty-five to ninety, and few even higher than ninety."*

* "Hydraulics, Dynamics, etc.", p. 211, James Emerson, Williamsett, 1894.

Mr. Groat has asked for more definite information regarding the efficiency test of the Gardner's Fall plant. One cannot do better than quote the description of it which appears on page 45 of *Bulletin 109* of the S. Morgan Smith Company.

"Measurements of the effective head on the wheels were made by means of piezometers at different points. One of these at about the middle of each scroll was agreed upon to represent fairly the elevation of the water above the wheels. The tail-water below each wheel was observed by a float gauge below the power house where the draft-tubes discharge into a short tail-race. The output in kilowatts was measured by meters tested in advance under an agreement that the results indicated would be accepted by both parties.

"Measurements of water were made by the Greenfield Company in the canal above the power house and by the S. Morgan Smith Company in the tail-race below, with some check measurements by the former in the tail-race.

"The actual losses of head from the canal to the river at the end of the tail-race were not only reasonable and less than expected, but indicated that the wheels were getting, as expected, practically the gross head between the two levels.

"The only unfavorable condition was the strong wind blowing down the canal in the direction of the power house at the time of the best results at part gate.

"A. T. SAFFORD, *Engineer*".

The head was taken as the difference between head and tail-water readings, and no correction was made for the velocity head of the draft-tube discharge. Fteley-Stearns current meters were used. Although the results of this test are high, the most searching criticism has failed to find reason for doubting them.

Mr. Williams states that, due to his draft-tube designs, a gain of 7½% in efficiency has resulted over that obtained with wheels equipped with the "standard type" of draft-tube. If by "standard type", Mr. Williams means a long straight tube, or even a curved tube with a long straight section, the writers cannot agree. Can there be a better draft-tube than the long flaring four-diameter tube generally used at Holyoke? Are not all the patent draft-tubes of to-day merely modifications designed to secure nearly as good results in a shorter over-all distance?

The writers agree with Mr. Williams regarding the accuracy of the Holyoke Flume. An investigation made in 1900 by one of them led to the conclusion that for wheels discharging more than 200 sec-ft., the error in favor of the wheel was somewhat more than 2 per cent. There is no question regarding the comparative accuracy of the testing flume, but the results are not absolute. It would seem that this fact should be more thoroughly appreciated at the present time when so much attention is being paid to tests in place.

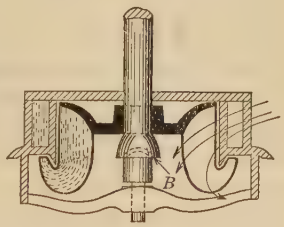

FIG. 62.—OBENCHAIN TURBINE, 1871.

Mr. Floyd A. Nagler quotes Robert E. Horton, M. Am. Soc. C. E., who says that McCormick made probably the greatest single achievement when he extended the buckets downward and outward. Although McCormick was responsible for much valuable development work along these lines, he merely enlarged on the design of the Obenchain "Little Giant" (Fig. 62), which preceded the "Hercules" by about five years.

Regarding the ridiculous efficiency claims of some of the early water-wheel makers, Mr. Nagler states that the writers' figure for the efficiency of the Wynkoop wheel should be 135% rather than the 175% given. The following is a quotation from one of L. D. Wynkoop's circulars (undated), which is also reproduced in Emerson's 1894 edition, on the page facing that referred to by Mr. Nagler:

"CERTIFICATE.

"From the experiments performed with the Wynkoop wheel in the foundry of Messrs. Clapp and Hamblin of this city, I find it to utilize more than 175 per cent. of the absolute weight of the water used; probably nearer 200. This I regard as no violation of the principle laid down in our natural philosophy, *viz.*, that no wheel can be invented which will utilize 100 per cent., as the wheel in question is not a single one, but such a combination of wheels, as cannot fail to give a vast increase of power.

"I. C. COCHRAN, Principal of Owosso Union School.
"HENRY GOULD, Millwright, Owosso City."

The "Little Giant" turbine mentioned by Mr. Wood consisted of two mixed-flow runners, cast back to back, one discharging upward and the other downward. Mr. Wood's photograph (Fig. 35) is of the Canadian model of this wheel in which the lower runner was omitted and only the upward discharging runner was used. The Reynolds wheel, made in Oswego, N. Y., was of the same type, except that instead of having two separate runners fastened together, it had a single runner with both upward and downward discharge. A wheel made by Cushman of Hartford, Conn., was also constructed on the same principle.

Mr. Horton disagrees with the writers on several points. Perhaps it will be best to treat these in the same order in which they appear.

The writers have made as thorough an investigation of the records of the Patent Office as was possible. Unfortunately, this could not be done until after the paper was prepared, but the results are in part incorporated in the discussion. Nothing was found which in any way changed the views of the writers with regard to the development of the water-wheel. In fact, this study was hardly necessary, for nothing of importance was found that was not already on file in the library of the Proprietors of the Locks and Canals. At the time that Mr. Francis became interested in the principle of inward flow, he made a thorough investigation of Howd's inventions and secured copies of the patents. At about this same time, Mr. Boyden was collecting information regarding the wheels of the period, and thanks to these two men, the writers found themselves provided with a wealth of contemporary information. Mr. Horton will notice that, with the exception of Johnson's wheel, every one that he mentions in Table 3 has been discussed by the writers. Leffel's wheel of 1845, as mentioned in Table 3, was a mere modification of a Jonval runner and was of no importance in the history of water-wheel development. This wheel did not have guide vanes, but was set in a scroll case.

Mr. Horton is apparently under the impression that the modern wheel ("large capacity, high-speed, mixed inward, and axial flow turbine") depends for its action on a volute case or scroll. This is far from the truth. Under the lower heads, a roomy open-flume setting is often much to be preferred, and

Holyoke tests are made with such a setting. It is granted that for structural reasons a scroll is of great advantage under high heads, but under these conditions the runner becomes low speed. He also states that "pivot guide gates" are one of the essential features of this type of wheel. This excludes all cylinder-gate wheels, such as the excellent "Hercules" Type D, which, although not of extremely high speed, is probably the most thoroughly American wheel made to-day.

Mr. Horton feels that sufficient appreciation of Howd is not included in the paper. On a closer reading he will find that Howd is given credit for the invention, but not for its development. The following is a quotation of the inventor's own words from an advertisement* in which, decrying the principle of inward flow, he states that outward flow is the panacea, thus reversing his former claims:

"A Recent Important Hydraulic Discovery! Highly valuable to millers, manufacturers and millwrights. Howd's latest improved Water Wheel! Combining all the excellent properties of his former wheel, the Reaction, Gilbert's and the Turbine Wheels, and embracing in addition thereto several new and efficacious principles in its construction and operation, rendering it far superior to any of them in respect to durability, expense and efficiency.

* * * * * * * *

"At the time the undersigned began his researches upon the subject, a Wheel well and generally known as the 'Reacting Wheel' was in most general use upon such streams; and having often witnessed the operation of this Wheel, it appeared to the undersigned, from a comparison of the head and quantity of water applied to it, with the power obtained, that it was susceptible of much improvement, or that a Wheel upon some entirely new principle or combination of principles, might be invented which, under the same circumstances, would be more efficient. Prompted by this impression, after much close investigation, and many trials and experiments, the undersigned in the year 1836, introduced to the public a Wheel now extensively known as 'Howd's Improved Water Wheel', of which the following is a brief description:

"The buckets, the impinging face of which are bounded by arcs of small circles, are placed between two flat rims, the upper of which is attached by arms to a perpendicular shaft. The water is admitted from without inwardly through shutes at right angles to the radius at the point of impingement, and the motion of the wheel is the same as the direction of the water.

"This improved Wheel, upon being tested, was found to excel in every respect, the 'Reacting Wheel', insomuch that it has been extensively adopted, and has until recently been superseding all others used under low or ordinary heads of water.

"But notwithstanding so much was gained by the Improved Wheel, the undersigned by a long series of scrutinizing observations, discovered several imperfections about it which, if removed or neutralized, would, in all probability, greatly enhance its utility.

"The imperfections are these: First, the water being admitted from without on all sides through shutes, the outward apertures of which are open to the volume of water in the flume, and directed at the same angle on the periphery of the Wheel, an artificial whirling or circumgyration of the water in volume in flume is produced, which tends to remove it by centrifugal action, from the entrance of the shutes, and consequently diminishes the pressure to a con-

* Published in the *Wayne Standard* (Wayne County, New York), of June 18th, 1842.

siderable extent, making a difference in the motion of the Wheel, that may be easily ascertained, other things being equal, to be proportionate to such diminution.

"Secondly.—The water is unphilosophically introduced upon the Wheel by being admitted from without, or upon the periphery. For a rotary motion of the Wheel tends to throw it back, so that in causing the water to perform such an unnatural function there is a conflict between the centrifugal force and the force produced by the pressure of the water; the result of which is that the power of the Wheel is less, by a quantity just equal to such centrifugal force, which is very considerable, operating as it does at every point of the periphery.

* * * * * * * *

"Sixthly.—The water being discharged inwardly it is prevented, in cases of low heads, from flowing off readily, and in consequence thereof accumulates and rises above the wheel, and around the shaft several inches, the effect of which is equivalent to a reduction of the head an equal number of inches.

* * * * * * * *

"Being fully convinced of the existence of the above mentioned imperfections, and incited by the hope of being successful in his attempts to remedy them, the undersigned carried out another protracted course of arduous experiments, testing every principle both by theory and practice, and finally in the month of September last, came before the public with another Wheel, designated and hereafter to be known as 'Howd's Latest Improved Water-Wheel', differing from his first Wheel mainly in these particulars:

"First.—The water being admitted from within, the artificial whirling or circumgyration before alluded to, tends to move it centrifugally toward the periphery; or in other words, increase the pressure at the point of impingement.

"Secondly.—In consequence of the admission of the water from within, it is permitted to pursue its natural centrifugal force acting at the periphery.

* * * * * * * *

"Lastly.—This Wheel when running to the right, has all the advantage of that natural and peculiar cirmumgyratory motion of the water, resulting as it is supposed from the diurnal revolution of the earth, and which never assists the old Wheel, but always retards it.

"The undersigned has obtained Letters Patent for his improvement, and is now ready to dispose of Individual and Territorial Rights, to use the same. Numerous certificates, recommendatory of his latest Improved Wheel might be published, but rather than rely upon them for a proper public appreciation of its intrinsic merits, he has chosen to present for consideration the scientific principles involved in its construction, and to rely upon the opinion which candid, skillful and intelligent Mill-Wrights will pronounce, after such principles shall have been by them fully and carefully examined. If its inherent merits 'unpuffed' will not recommend and sustain it; it must sink 'unpuffed' into oblivion.

"The public are cautioned to bear in mind the distinction between 'Howd's Improved Wheel' and 'Howd's Latest Improved Wheel'.

"Postage must be paid on all communications by mail, or they will not be taken from the Post-Office.

"Samuel B. Howd."

"Newark, Wayne Co., N. Y., May 12, 1842.

The advertisement shows a wood cut of a crude Fourneyron turbine.

Mr. Horton's Fig. 36 shows the Howd wheel of 1851. This was a later model, and might well have been modified in accordance with Francis' improvements. Fig. 63 shows the Howd wheel as it was made in 1847. The diagram is an exact copy drawn from measurements made by Mr. Francis on one of

these wheels in June, 1847. The bucket outline is copied from a full-sized sketch accompanying these measurements. This wheel is basicly different from the one shown by Mr. Horton. For a 5-ft. wheel, the total guide area of discharge at the narrowest point was 1.51 sq. ft. and the same area for the buckets was 5.00 sq. ft. Advertisements of that time show the wheel set above tail-water and discharging into the air. Later, it was submerged, but apparently it was not in the early models. It is evident from these two facts that this wheel must have been primarily an impulse wheel. It was supposed to operate at a relative velocity of from about 0.33 to 0.50.

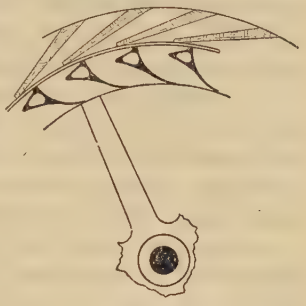

FIG. 63.—HOWARD WHEEL, 1847.

Howd was the inventor of the wheel which inspired the Francis turbine, but did he appreciate it? Francis evolved, from Howd's discarded invention, the genesis of the modern turbine. To which man should the greater credit be given?

Mr. Horton does not feel willing to accept the "Family Tree" of the modern turbine. Apparently, he feels that the "volute vortex" turbine suddenly sprang into being. Yet he admits earlier in his discussion that the turbine was the result of gradual evolution. He says, "Again, the American, Hercules, Victor, and Sampson turbines were off-shoots of the Obenchain and McCormick wheels * * *." The first "American" turbine was patented in 1859, twelve years before the Obenchain "Little Giant". Otherwise, Mr. Horton's statement is quite correct, and a closer examination of the paper will show that this is precisely what the "Family Tree" and text demonstrate.

The Parkers patented the draft-tube in 1840 and this preceded Jonval, as his wheels were not built until 1841.* Granting that the modern flaring draft-tube was suggested by Boyden's diffuser, it is difficult to conceive how the diffuser could have suggested the "outlet cone".

In his Table 4, Mr. Horton shows the marine propeller as a development of the Austin wheel by way of the current meter. The date of the Austin wheel is not known, but it probably was not earlier than 1860. The records of the Patent Office show no record of any water-wheel patent issued to any one by the name of Austin up to 1873. The first screw current meter probably was Woltmann's "Moulinet". The earliest record of this found by the writers is a paper written in 1847 by Baumgarten,† in which it is implied that the meter is about ten years old. John Stevens' steamboat of 1804 had a propeller, and the use of a screw propeller was suggested in 1680 by Hooke.‡

Mr. Horton states that the modern wicket-gate is a recent European invention. It is merely the Leffel gate of 1862 made stream line. Can this be called an invention?

He does not feel that the Truax wheel is a good illustration of the early propeller wheels, and thinks the Austin wheel more suitable. The writers have

* J. Buchetti, "Les Moteurs Hydrauliques Actuels", p. IX.

† *Annales des Ponts et Chaussees*, 1847.

‡ Appleton's "Cyclopedia of Applied Mechanics", p. 719, New York, 1880.

a four-bladed Truax wheel and a two-bladed Austin wheel, both loaned by Mr. Kinne. Except for the difference in the number of the blades, the wheels are practically identical. The Austin wheel was also made with three blades, and the Truax with two, three, four, and five blades. Some of the Truax wheels had the saw-teeth buckets around the rim, that were omitted in others.

In preparing this paper, the writers have particularly had in mind small and medium-sized developments under comparatively low heads, in which field their experience has largely been. They have not attempted to trace the truly remarkable increase in size of modern units and the increasing use of the higher heads. This subject might well form a chapter by itself.

It is realized that this increase in size has been an essential step in the development of the modern hydro-electric system. The writers feel that the advance has been basicly structural and mechanical rather than hydraulic. Full credit, however, is due to those designers who, in recent years, have done so much to increase the speed and power of the mixed-flow turbine.

It is believed that the successful operation of a unit in place depends on the correct design of the wheel and its setting, based on proper velocity and velocity distribution and on sound hydraulic principles, rather than on any particular shape, patent, or popularity.

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Paper No. 1504

SURGE TANKS

By B. F. Jakobsen,* Assoc. M. Am. Soc. C. E.†

With Discussion by Messrs. F. G. Baum, P. Wahlman, Charles P. Dunn, Minton M. Warren, and B. F. Jakobsen.

Synopsis

The distinction between surge tanks for acceleration and deceleration is shown, and likewise the distinction between the requirements for medium high-head plants and for high-head plants. Tests of the Kerckhoff surge tank are presented, and the computed values are shown to agree closely with the tests. The tests show the rapid dampening of the surge that took place. The mathematical formulas are developed in the Appendices.

Introduction

With the growth of the size of power plants has come the increasing length of closed conduits. Where such conduits are used in conjunction with turbine installations, it becomes advisable, and, in some cases, necessary, to use surge tanks in order to prevent excessive water-hammer when the turbine rejects load.

In this type of power plant, it also becomes necessary to provide water storage near the plant, so that the load on the plant can be increased as fast as modern operation requires. A surge tank designed for this last condition will also relieve surges or prevent them from entering the long, closed conduit.

General

A typical surge-tank installation (Fig. 1) shows the tank located at the lower end of a long, closed conduit and at the top of the penstock. The object of the surge tank is to relieve surges which travel up the penstock when the turbine is rejecting load and to furnish water to the turbine when the load is

* Cons. Hydr. and Elec. Engr., Fresno, Calif.
† Now M. Am. Soc. C. E.

increased, since the gradient of the long conduit seldom suffices to provide the maximum acceleration needed for satisfactory regulation and operation of the plant.

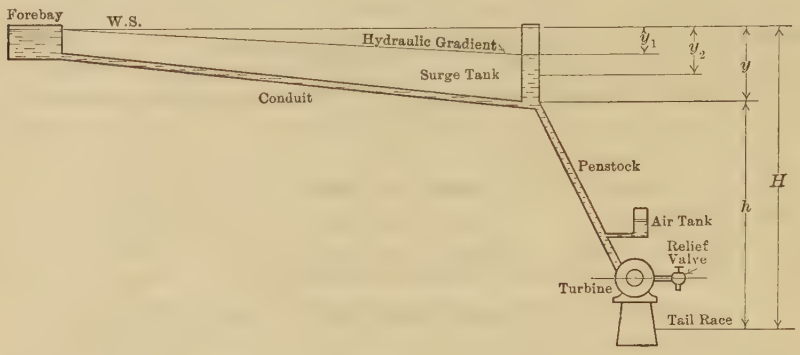


FIG. 1.

Surge tanks may be divided into two general classes, depending on the main object to be obtained:

I.—Surge tanks designed so as to allow the plant to take on load rapidly, and to spill on load rejection of no consequence.

II.—Surge tanks designed primarily to prevent spilling, on load rejection.

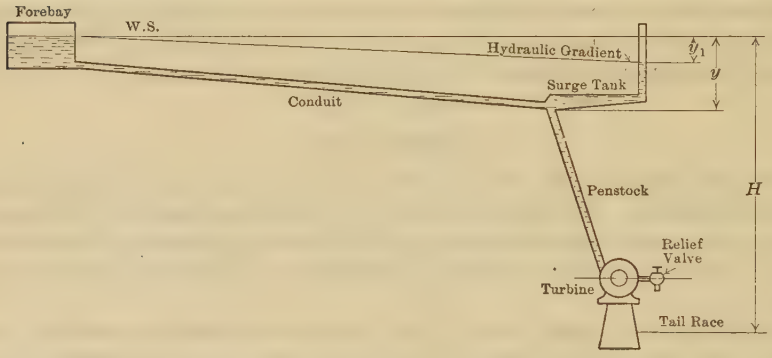


FIG. 2.

Case I.—Rapid Acceleration, as Shown in Fig. 2.—The water storage lies just above the lower end of the long, closed conduit. With a certain load on the plant, the drawdown is y_1 ft., as indicated. If the load is rapidly increased, the surge tank must supply the additional water, and the level of the water in the surge tank will be quickly lowered to the top of the storage chamber. In this design, the maximum accelerating head is made available for the long conduit practically from the start, and this, therefore, permits the use of the smallest possible surge tank. If, however, y is a considerable part of H, as it is likely to be with medium high-head plants having long conduits, the

head on the turbines is lowered at the time it is most needed, and the lowering of the head in the surge tank operates to decrease the total available output of the plant. To overcome this disadvantage, the storage must be somewhat distributed, as discussed subsequently.

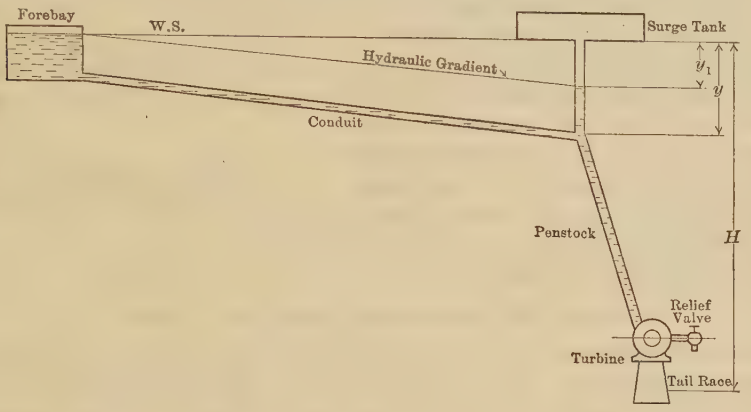


FIG. 3.

Case II.—Rapid Deceleration, as Shown in Fig. 3.—It is evident that with Case II, the storage should be placed as high as possible without additional cost, in order to reduce its volume to the least possible. The type of surge tank, shown in Fig. 3, will be only of small assistance to the plant when the load is increased. The combination of Fig. 2 and Fig. 3 would have the advantages of both, but the disadvantage, due to the lowering of the head on the turbine when the load is increased, will still remain. With medium high-head plants, or where y, Fig. 1, is a considerable part of H, it will generally be advisable to distribute the storage so that the two types of surge tanks develop the shapes shown in Fig. 4 and Fig. 5.

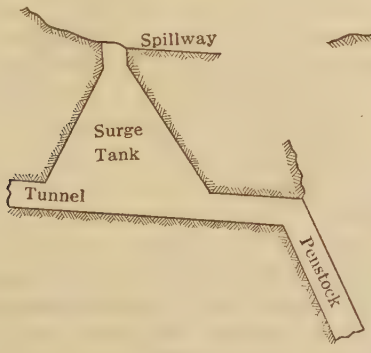


FIG. 4.

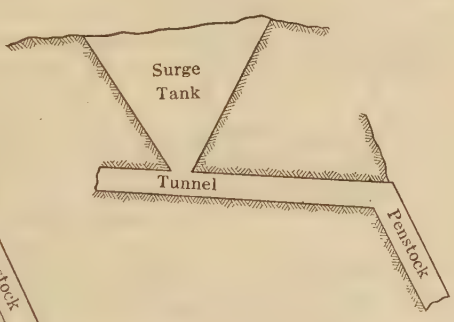


FIG. 5.

SURGE TANK FOR HIGH-HEAD PLANTS

With plants where y is a considerable part of H, the surge tank shown in Fig. 2 will be the most advantageous. It may be a circular conduit, built slightly above the roof of the tunnel and inclined just enough to insure proper

operation. This conduit may be a short section of tunnel or of pipe. Fig. 6 shows a vertical section of such a surge tank, which tank was designed by the writer in connection with the plant of the Homestake Mining Company in South Dakota, and was built by F. G. Baum, M. Am. Soc. C. E., A. L. Wilcox, Assoc. M. Am. Soc. C. E., acting as Construction Engineer.

The computations for such a tank are comparatively simple, since it may be assumed that the water surface is instantly lowered and, therefore, the head, y, operating on the water in the tunnel or long conduit may be assumed to be constant without introducing any great error. This assumption simplifies the computations, as indicated in Appendix II.

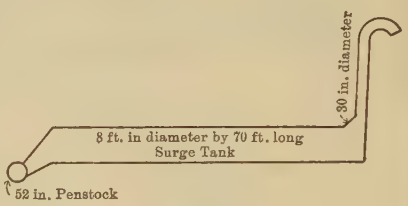


FIG. 6.

SURGE TANK FOR MEDIUM HIGH-HEAD PLANTS

Fig. 4 represents the design proposed by the writer for plants where y is a considerable part of H. This shape of tank has the advantage that its area increases as the hydraulic gradient is lowered, that is, as the maximum accelerating head decreases, and a surge tank of this general shape, therefore, has its storage where it is most needed.

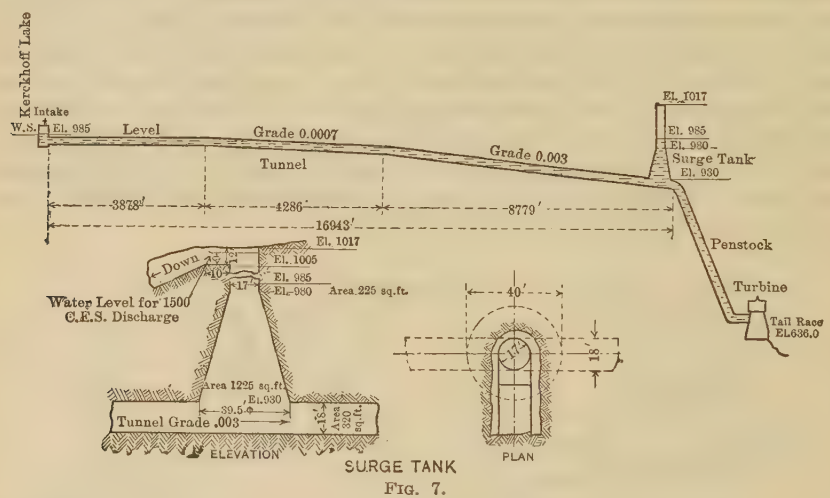


FIG. 7.

Where the surge tank is excavated in solid rock, as at the Kerckhoff power plant, the cost of excavating is practically independent of the shape of the tank, and one similar to that shown in Fig. 4 will give the least excavation and, therefore, the lowest cost. If the tank has to be built of reinforced concrete, the shape shown in Fig. 4 may cost more than that shown in Fig. 5 for equal volumes, and the problem is complicated by the additional fact that the shape of the tank influences its cost.

The Kerckhoff surge tank, shown on Fig. 7, is 17 ft. in diameter at the top and 39.5 ft. in diameter at the bottom. These dimensions were decided

on after a few trial computations with various assumed load changes, and after deciding, in consultation with the Operating Department, the extreme load change that the plant must meet. The writer knows of no general method of determining the most economical shape of tank; this will vary with the assumed load change. The tank giving the smallest cost for the extreme load change it is able to take care of, will generally be the one to select. The smaller diameter should be selected so as to give satisfactory spilling conditions when load is rejected by the plant.

Kerckhoff Surge Tank Test

The Kerckhoff surge tank and the hydraulic data for the conduits are shown in Fig. 7. The tunnel, which is about 18 by 18 ft., with a slightly arched roof, has an average area of 320 sq. ft. The power plant has three turbines, each with a maximum rating of 15 000 h. p., at 315-ft. effective head, connected to 14 200-kv-a. generators. The three penstocks, which are 8 ft. in diameter at the top, tapering to 7 ft. at the bottom, average in length about 930 ft. This plant, built by the San Joaquin Light and Power Corporation, was put on the line in August, 1920. R. C. Starr, M. Am. Soc. C. E., is Construction Engineer, and the writer is Designing Engineer for the Company. L. O. Wolcott, Assoc. M. Am. Soc. C. E., made most of the computations for the surge tank.

To determine how closely the computations check with tests, several load-on and load-off tests were made on October 16th, 1921, under the supervision of E. A. Quinn, General Superintendent of Operation.

It was intended to obtain the surge tank readings by means of a Bristol gauge which was connected to a bulb placed in the bottom of the surge tank. During the first test, however, the bulb was broken and could not be replaced in time for the test. It was necessary, therefore, to measure the water surface in the surge tank by a float tied to the end of a steel tape. As one of the units was connected to the system of the Pacific Gas and Electric Company, the load on it could not be varied. The other two units were used in the test, the load being transferred as needed between the Kerckhoff plant and the steam plants at Bakersfield and Midway, Calif.

Two tests with load on and two tests with load off were made. During the second load-on test, the tape broke so that the data are not complete. Table 1 gives one test for load on, and Table 2, one test for load off. The watt-meters in the power house were read by the power-house crew, under the supervision of Mr. Goldsworthy, Superintendent of the Plant, and the surge-tank readings were made by L. J. Moore, Executive Engineer. The load was transferred from or to the Kerckhoff power house as fast as possible without too great variations of the frequency of the system.

The readings of the elevation of the float in the surge tank were taken at intervals of 15 sec., and are shown on Fig. 8, on which is also plotted the load on the plant. The first small surge is probably due to the rapid increase of load in the beginning, followed by a less rapid increase. As noted in Table 1, the load on Generator No. 3 was read only at the beginning and at the end of the test. In plotting the load, it was assumed that

this decreased roughly in the same proportion as the load on the other two units increased. In order not unduly to complicate the check computations, which are plotted on Fig. 9, the load was assumed to increase uniformly from 9 000 to 27 850 kw. in 70 sec.

TABLE 1.—Load-On Test.

Time, in seconds.	Load on Generators, in Kilowatts.		
	No. 1.	No. 2.	No. 3.
0	0	0	9 000
15	1 000	2 500	No reading
30	4 000	6 500	" "
45	6 000	8 500	" "
60	10 000	8 000	" "
15	9 500	8 500	" "
30	10 500	9 700	" "
45	10 500	10 250	" "
120	9 600	10 250	8 000

TABLE 2.—Load-Off Test.

Time, in seconds.	Load on Generators, in Kilowatts.		
	No. 1	No. 2.	No. 3.
0	10 000	7 500	8 500
5	8 000	6 500	No reading
10	4 000	3 000	" "
15	500	0	" "
20	0	0	" "
0	0	0	8 500

The quantity of water was estimated as follows: With Generator No. 3 carrying 9 000 kw., and Generators Nos. 1 and 2 carrying no load, but operating at synchronous speed, the elevation of the water in the surge tank was 981.5 ft. and in the tail-race, 636.0 ft., thus giving an effective head of 345.5 ft. Fig. 10 shows that the water required at no load is about 16% of that required at full load, but 17% was assumed, to allow for water taken by the exciter, the load on which varied from 67 to 109 kw., and cooling water for the step-bearings and the transformers. The maximum load of one unit is approximately 18 700 h. p., so that the load required for one unit is about 3 180 h. p., or 162 cu. ft. per sec., for the two machines. The water required by Unit No. 3, carrying 9 000 kw., with a turbine efficiency of 96% and a friction head of 5.0 ft. in the penstocks, was 374 cu. ft. per sec., so that the total water at the start was 536 cu. ft. per sec. Likewise, the water required at the close of the test, with 27 850 kw. on the plant and with the water in the surge tank at Elevation 970.2, was determined as 1 182 cu. ft. per sec. This quantity was incorrectly estimated as 1 145 cu. ft. per sec., and Fig. 9 was computed before the error was discovered, but as this error is only about 3%,

it was thought not worth while to re-compute this test, as the surge tank was excavated from solid rock, and its dimensions most likely vary more than 3 per cent.

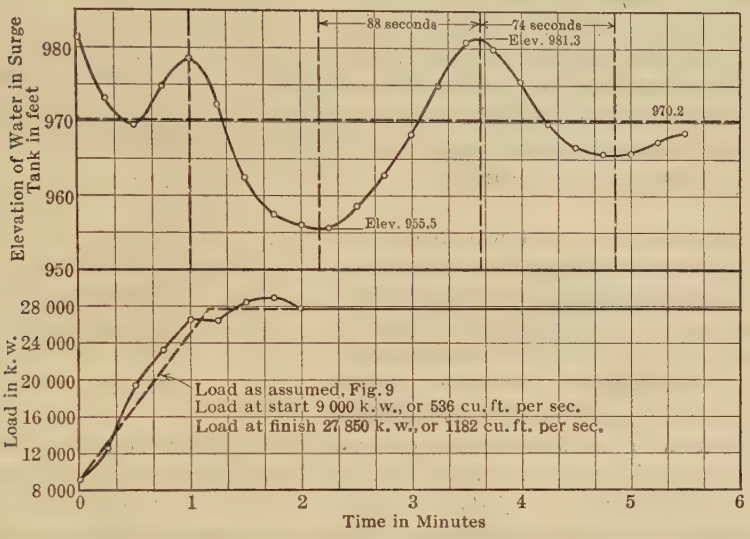


FIG. 8.

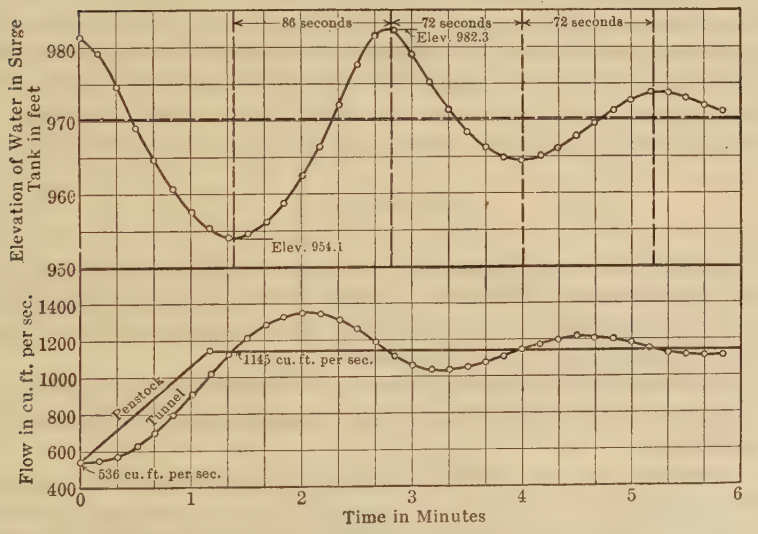


FIG. 9.

The friction loss in the tunnel was the difference in the elevations of the water in the surge tank and of Kerckhoff Lake which, throughout the test, was 984.2. For 536 cu. ft. per sec., the elevation of the water in the surge tank was 981.5, and for 1 145 cu. ft. per sec. (should have been 1 182 cu. ft. per

sec.), it was 970.2, thus indicating a friction loss in the tunnel of 2.7 ft. and 14 ft., respectively. This corresponds roughly to Kutter's $n = 0.032$. A comparison of Figs. 8 and 9 shows that the actual surge was slightly less than the computed surge, probably due to the fact that the load as assumed did not coincide with the actual load and, also, because the surge tank may be slightly larger than that called for in the design.

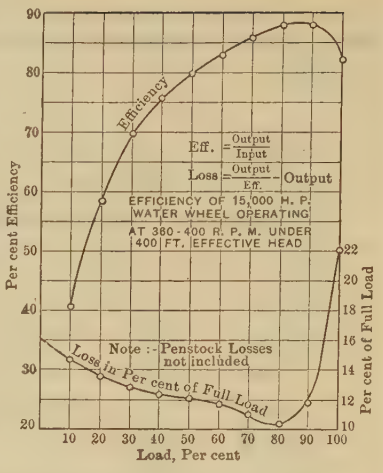


FIG. 10.

At the start, the load was 26 000 kw. Fig. 11 shows the computations. The time, as taken by a stop-watch, from the start of the test until the water reached its highest elevation at 1008.5, or 3.5 ft. above the spillway lip of the surge tank, was 25 sec., which agrees within 1 sec. with the computations. The elevation of the first downward surge was not taken on account of the rapid motion of the water surface, but Mr. Moore, who took these measurements, stated at the time of the test that he thought the water was at about Elevation 968, whereas the computations give 963. A measurement on the second downward surge was taken, but the time was not noted. The time from the second upward surge to the second downward surge was taken by a stop-watch as 60 sec., whereas the computations show 62 or 63 sec. The somewhat larger computed surges were due to the operation of the relief valves at the turbines. These valves have a discharge capacity of 50% of the full load discharge of the turbines, so that when fully open, each relief valve would discharge about 275 cu. ft. per sec. No means of measuring this discharge was available.

APPENDIX I

The general differential equations are given in Appendix III. The two resulting simultaneous differential equations cannot be integrated, and their use requires computations to be made for time intervals of from ¼ sec. to 1 sec. apart, depending on the curvature.*

It is believed that the following method, which allows time intervals of 10 sec. and more to be used, offers a much simpler and a sufficiently close approximation.

It is assumed that for sufficiently small time intervals, t, the quantities involved are linear functions of t. The velocity head in the long conduit has been neglected, as in most cases it is small, and the accuracy of the whole investigation depends on the friction loss in the tunnel which, in general, cannot be closely estimated beforehand. If, in order to be on the safe side, the friction loss in the tunnel is taken large enough, the velocity head may be assumed to be included in the friction head.

* "Water Power Engineering", by Daniel W. Mead, Appendix C.

The flow in the penstock may vary in any manner desired, so that, if desirable, the action of the governor may be included in the computations. In general, such a refinement is unnecessary as the assumed maximum load changes are determined approximately and are largely a matter of personal judgment. Also, most of the other quantities that influence the performance of the surge tank, are not known accurately at the time the tank is designed. As a rule, it will be sufficiently accurate to assume the flow in the penstock as increasing uniformly from a starting value to the final value, but any assumption as to load variation may be made.

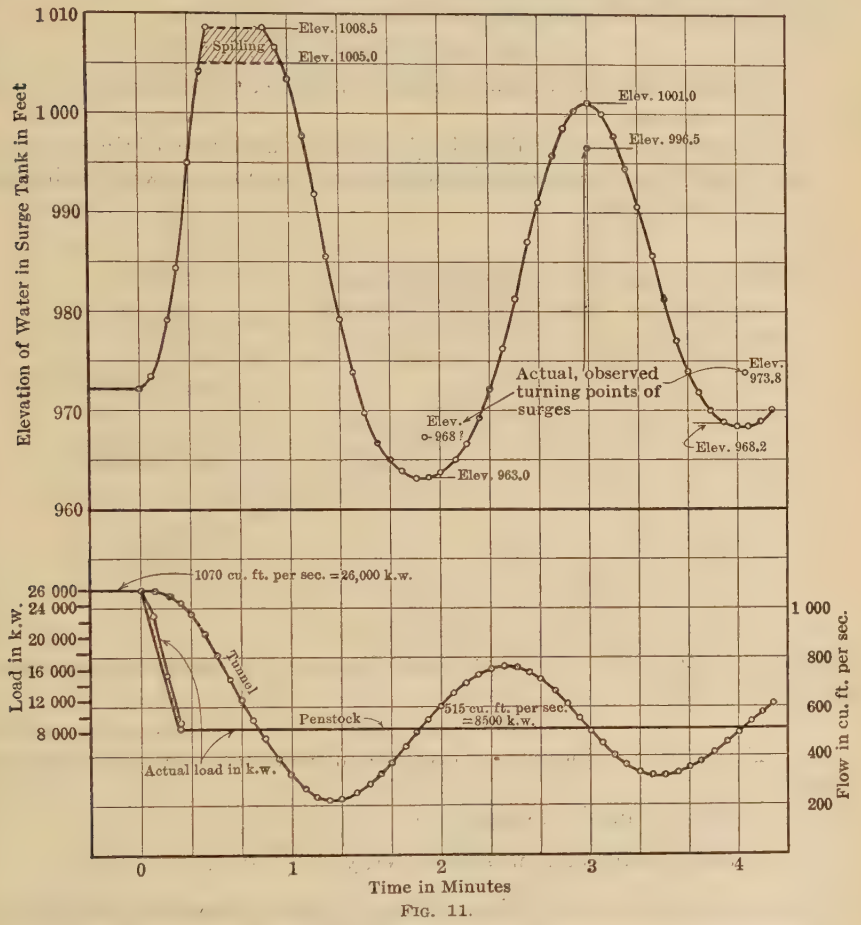


FIG. 11.

In surge-tank design, the assumption is often made that the load change takes place instantaneously or during a very short interval. This is not generally correct, and the following formulas were developed for any load change. In the case of Kerckhoff surge tank, it was the opinion in the Operating Department that, in case of trouble, a certain part of the system load is lost and most of this comes back gradually within the next few minutes. The

surge tank was designed, therefore, for a uniform load change from no load (no cubic feet per second) to full load in 3 min. It was also computed for smaller instantaneous changes of load, such as, in the opinion of the members of the Operating Department, would or might occur.

Surge Tank Calculations (Fig. 12).

Q_p = cubic feet flowing through penstock, in Δt sec., always $\gtreqless 0$.

Q_t = cubic feet flowing through tunnel, in Δt sec., positive when flowing toward power plant.

Q_s = cubic feet flowing from or to surge tank, in Δt sec.

$$Q_s = Q_p - Q_t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

q_p = cubic feet per second flowing in penstock at beginning of time, Δt sec.

q_t = cubic feet per second flowing in tunnel at beginning of time, Δt sec.

q_s = cubic feet per second flowing from or to surge tank at beginning of Δt sec.

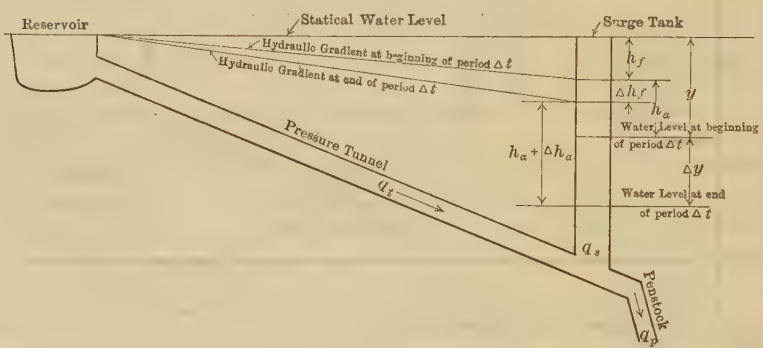


Fig. 12.

Therefore, Q_s is positive when water flows out of the surge tank, and *vice versa*.

$$q_s = q_p - q_t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Δq_p, Δq_t, and Δq_s increase or decrease in cubic feet per second during Δt sec.

A_t = area of tunnel, in square feet.

A_s = area of surge tank, in square feet, $y + \frac{\Delta y}{2}$ ft. from the top.

h_a = accelerating head acting on water in tunnel at beginning of Δt sec.

h_a is positive when the water in the tunnel is being accelerated while flowing toward the power plant.

Δh_a = increment of h_a during the time, Δt sec.

y = drawdown in surge tank at beginning of Δt sec., positive, when measured downward from zero position.

$y + \Delta y$ = drawdown at surge tank at end of time, Δt sec.

Therefore,

$$A_s \times \Delta y = Q_s \quad \ldots\ldots (3)$$

h_f = loss of head in tunnel due to friction and velocity heads, at beginning of time, Δt.

Δh_f = increment of h_f during time element, Δt.

$$y = h_a + h_f \quad \ldots\ldots (4)$$

a_i = acceleration of water in tunnel at beginning of time, Δt.

a_f = acceleration of water in tunnel at end of time, Δt.

a = average acceleration of water in tunnel during time interval, Δt.

g = 32.2 ft. per sec. per sec.

L = length of tunnel from forebay to surge tank, in feet.

In the time, Δt:

q_p becomes $q_p + \Delta q_p$, etc.

h_a becomes $h_a + \Delta h_a$, etc.

Friction Head.—

At beginning of time, Δt,

h_f corresponds to q_t.

At end of time, Δt,

$h_f + \Delta h_f$ corresponds to $q_t + \Delta q_t$.

Average value,

$h_f + 0.5\ \Delta h_f$ corresponds to $q_t + 0.5\ \Delta q_t$.

Accelerating Head.—

At beginning of period, Δt,

$$h_a = y - h_f.$$

At end of period, Δt,

$$h_a + \Delta h_a = y + \Delta y - h_f - \Delta h_f.$$

The average accelerating head, therefore, assuming that h_a, h_f, and y are linear functions for small values of Δt, is:

$$h_a + 0.5\ \Delta h_a = y - h_f + 0.5\ (\Delta y - \Delta h_f) \quad \ldots\ldots (5)$$

Acceleration.—

$$\left.\begin{aligned} a_i &= \frac{d\,v}{d\,t} = \frac{h_a\,g}{L} \\ a_f &= (h_a + \Delta h_a)\,\frac{g}{L} \\ \text{average acceleration,}\quad a &= (h_a + 0.5\ \Delta h_a)\,\frac{g}{L} \end{aligned}\right\} \quad \ldots\ldots (6)$$

Flow to or from Surge Tank, in Time, Δt.—

$$Q_s = \Delta y\, A_s = (\Delta h_a + \Delta h_f)\ A_s \text{ cu. ft.} \quad \ldots\ldots (7)$$

Flow Through Tunnel, in Times, Δt.—The increase of flow through the tunnel in time, Δt, is:

$$\Delta q_t = a\ \Delta t \times A_t \text{ cu. ft. per sec.,}$$

or,

$$\Delta q_t = A_t (h_a + 0.5\ \Delta h_a) \frac{g}{L} \Delta t \text{ cu. ft. per sec} \quad \ldots\ldots\ldots\ldots (8)$$

General Equations.—From Equation (1):

$Q_p = Q_t + Q_s$ cu. ft. flowing through the penstock, in Δt sec.

Assuming that the time interval, Δt, is chosen so small that the changes in q_p, q_t, and q_s, may be considered as linear, so that,

$$\left.\begin{aligned} Q_p &= \left(q_p + \frac{\Delta q_p}{2}\right) \Delta t \\ Q_t &= \left(q_t + \frac{\Delta q_t}{2}\right) \Delta t \\ Q_s &= \left(q_s + \frac{\Delta q_s}{2}\right) \Delta t \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Then,

$$Q_p = (q_p + 0.5\ \Delta q_p)\ \Delta t = (q_t + 0.5\ \Delta q_t)\ \Delta t + Q_s \text{ cu. ft.}$$

Substituting from Equation (8) for Δq_t and from Equation (7) for Q_s:

$$Q_p = q_t \Delta t + 0.5\ A_t (h_a + 0.5\ \Delta h_a) \frac{g}{L} \Delta t^2 + \Delta y\ A_s \text{ cu. ft.}$$

As, from Equation (4):

$$\Delta y = \Delta h_a + \Delta h_f$$

$$Q_p = q_t \Delta t + 0.5\ A_t h_a \frac{g}{L} \Delta t^2 + 0.5\ A_t \frac{g}{L} \frac{\Delta h_a}{2} \Delta t^2 + A_s \Delta h_a + A_s \Delta h_f$$

$$Q_p = q_t \Delta t + 0.5\ A_t h_a \frac{g}{L} \Delta t^2 + A_s \Delta h_f + \Delta h_a \left(0.5\ A_t \frac{g}{L} \frac{\Delta t^2}{2} + A_s\right)$$

Arranging:

$$\Delta h_a = \frac{Q_p - q_t \Delta t - 0.5\ A_t g\ h_a \Delta t \frac{2}{L} - A_s \Delta h_f}{A_s + 0.25\ A_t g \Delta t \frac{2}{L}} \text{ ft.} \quad \ldots\ldots (10)$$

and from Equation (8):

$$\Delta q_t = A_t (h_a + 0.5\ \Delta h_a)\ g\ \Delta \frac{t}{L} \text{ cu. ft. per sec.}$$

In Equation (10), the only unknown is $A_s \times \Delta h_f$; A_s is the average area of the surge tank at the water level during the period, Δt, or the arithmetical mean between the area of the water surface in the surge tank at the beginning and at the end of the period, Δt, and Δh_f is the increment of friction head during the time, Δt. In this case, A_s is considered to vary with y; if A_s is constant, the work is somewhat simplified. As A_s is known for the beginning of each period, Δt, the average value may be closely approximated; likewise, as q_t is known at the beginning of each period, h_f is known, and, therefore, Δh_f can be closely approximated. Therefore, assuming values for A_s and for Δh_f, Equation (10) gives Δh_a and Equation (8) gives Δq_t; and q_t and Δq_t determine Δh_f, which should check with the value assumed for Δh_f. As $\Delta y = \Delta h_a + \Delta h_f$, Δy can be determined, and as A_s depends on y and

Δy, A_s can be determined, and this should check with the value assumed for A_s.

The manipulation of Equations (10) and (8) can be best shown by an example. Referring to the load-on test previously mentioned, Fig. 13 has been plotted in order to have the values of A_s and h_f in a convenient form. The surge-tank area is plotted against the elevations and the tunnel friction loss against the cubic feet per second flowing in the tunnel. For Elevation 980, A_s is 225 sq. ft., and for Elevation 930, A_s is 1 225 sq. ft. Also, for

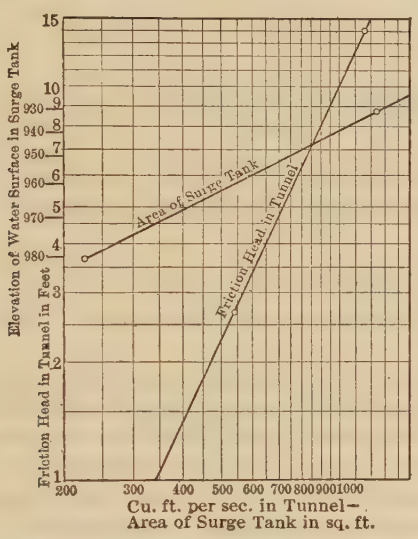


FIG. 13.

$q_t = 536$ cu. ft. per sec., the friction loss in the tunnel, h_f, is 2.7 ft., and for 1 145 cu. ft. per sec., it is 14 ft. $A_t = 320$ sq. ft., and length of tunnel, $L = 17\,000$ ft. The exact length of the tunnel is 16 943 ft., but 17 000 ft. was taken to allow for entrance losses, etc. The increase in q_p is from 536 to 1 145 cu. ft. per sec., in 70 sec., or 87 cu. ft. per sec. per each interval of 10 sec.

Introducing these constants, Equation (10) may be written:

$$\Delta h_a = \frac{Q_p - q_t \Delta t - 30.3\, h_a}{A_s - 15} - \Delta h_f \text{ ft.}$$

Equation (8) becomes:

$$\Delta q_t = 6.06 \left(h_a + \frac{\Delta h_a}{2} \right) \text{ cu. ft. per sec.}$$

(1) $\Delta t = 10$ sec.;* $h_a = 0$; $h_f = 2.7$ ft.; $y = 2.7$ ft.

$$Q_p = 10 \left(536 + \frac{87}{2} \right) = 5\,795 \text{ cu. ft.}$$

Assume:

$$\Delta h_f = 0;\ A_s = 225 \text{ sq. ft.},$$

then,

$$\Delta h_a = \frac{5\,795 - 5\,360}{210} = 2.08 \text{ ft.}$$

$$\Delta q_t = 6.06 \times 1.04 = 6 \text{ cu. ft. per sec.}$$

* A check computation with $\Delta t = 5$ sec. was made; this gave the same result as the computations made with $\Delta t = 10$ sec.

Therefore, $y = 2.7 + 2.08 = 4.8$ ft.; elevation of water surface in surge tank $= 984.28 - 4.8 = 979.4$ ft. The assumptions that $\Delta h_f = 0$ and $A_s = 225$, check (see Fig. 13).

(2) $\Delta t = 10$ sec.; $h_a = 2.1$ ft.; $h_f = 2.7$ ft.; $y = 4.8$ ft.

$$q_t = 542 \text{ cu. ft. per sec.}$$

$$Q_p = 10\left(536 + 3 \times \frac{87}{2}\right) = 6\,660 \text{ cu. ft.}$$

Assume:

$$\Delta h_f = 0.2 \text{ ft.}; \quad A_s = 275 \text{ sq. ft.},$$

then,

$$\Delta h_a = \frac{6\,660 - 5\,420 - 30.3 \times 2.1}{260} - 0.2 = 4.34 \text{ ft.}$$

$$\Delta q_t = 6.06\ (2.1 + 2.2) = 25.4 \text{ cu. ft. per sec.}$$

Therefore, the assumed value of Δh_f checks (see Fig. 13).

Also,

$$y = h_a + \Delta h_a + h_f + \Delta h_f, \text{ or } 9.34 \text{ ft.}$$

The elevation of the water in the surge tank is 974.86 ft.; therefore, A_s checks.

It will be found convenient to make a table and enter the variables for each step. This also facilitates the plotting which should be done as soon as each step is completed in order to act as a check on the computations. Care is necessary in order to observe the correct signs of Δh_a and Δh_f. It is evident, from Fig. 9, that when the flow in the tunnel equals the flow in the penstock, the surge curve is a maximum or a minimum.

The operations are identical when load is decreased, except for change in signs. The load-off condition is, in general, of much less importance than the load-on condition.

In order to show the difference in drawdown between a surge tank, with a larger area toward the bottom, and one of equal volume, but with its larger area toward the top, computations were made by inverting the Kerckhoff surge tank, as shown in Fig. 14, and using the same load changes as shown in Fig. 9, in which the water surface fell to Elevation 954.1, where $A_s = 640$ sq. ft. This was assumed as the area at Elevation 981.5 in Fig. 14, and, therefore, the two surge tanks contain the same volume between Elevations 981.5 and 954.1. It will be seen that the drawdown of Fig. 14 amounts to 41.1 ft., whereas, in Fig. 9 it amounts to only 27.4 ft. This shows the advantage of having the larger area of the tank toward the bottom.

APPENDIX II

The computations for a surge tank, as shown in Fig. 2, are considerably simplified, because y may be assumed as constant and, therefore, Δy is zero.

Then, from Equation (4):

$$h_a = y - h_f$$

and,

$$\Delta h_a = -\Delta h_f$$

and Equation (8) becomes:

$$\Delta q_t = A_t (h_a - 0.5 \Delta h_f) \times \frac{g}{L} \times \Delta t \ldots\ldots\ldots\ldots\ldots (12)$$

For the beginning of the period, $t = 0$, y and h_f are known and h_a is found from Equation (4); y is the distance down from the forebay level to the center of gravity of the enlarged part of the surge tank in Fig. 2. Assuming the

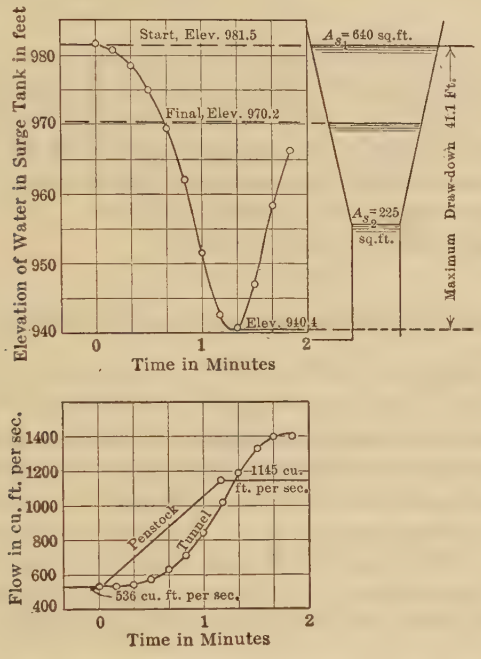


FIG. 14.

value for Δh_f, Equation (12) gives Δq_t, and the assumed value for Δh_f can then be checked. Δh_f can generally be closely approximated after the first or second step.

The volume required in the surge tank is then found, as shown in Fig. 15, by plotting the tunnel flow and the penstock flow with respect to time. The area between these two curves is the deficiency which the surge tank must supply. It is to be noted that whenever Δq_t, as determined from Equation (12), exceeds Δq_p, the rate of increase in the tunnel flow follows the rate of increase in the penstock flow and Equation (12) cannot be used until the instant Δq_t begins to fall below Δq_p.

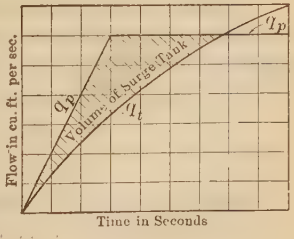


FIG. 15.

If Δt in Equation (12) approaches dt, and the flow in the tunnel is denoted by q instead of q_t, then from Equation (12):

$$dq = A_t \frac{g}{L} (y - h_f)\, dt \quad \ldots\ldots (13)$$

This can be integrated, if

$$h_f = f q^2, \text{ so that } y > f q^2 \quad \ldots\ldots (14)$$

Then, from Equation (12),

$$dt = \frac{L}{g A_t} \frac{dq}{y - f q^2} \quad \ldots\ldots (15)$$

and,

$$T = \frac{L}{2 g A_t \sqrt{y f}} ln \left[\frac{(\sqrt{y f} + f q_1)(\sqrt{y f} - f q_0)}{(\sqrt{y f} - f q_1)(\sqrt{y f} + f q_0)} \right] \text{sec.} \quad \ldots\ldots (16)$$

where q_0 is the flow at time, $t = 0$, and q_1 is the flow at time, T. When q_1 is the flow in the penstock after the completed load change, T is the time required for the flow in the tunnel to reach this value.

The surge tank volume is the difference:

$$Q_s = Q_p - Q_t.$$

Q_p is known from Equation (16); from Equation (15):

$$Q_t = \int_0^T q\, dt = \frac{L}{g A_t} \int_{q_0}^{q_1} \frac{q\, dq}{y - f q^2} = \frac{L}{g f A_t} ln \sqrt{\frac{y - f q_0^2}{y - f q_1^2}} \quad \ldots\ldots (17)$$

where q_1 is the flow in the penstock after the completed load change.

Equations (16) and (17) are not much more convenient than Equations (4) and (12), and, moreover, they are also subject to the condition that the rate of increase in the tunnel flow, dq, must be less than the increase in the penstock flow. The time, $t = 0$, must be counted from the moment this occurs. If this is not observed, a tank much too small may result.

In the case of the test of the Kerckhoff surge tank, shown in Fig. 9, ΔQ_p was 87 for $\Delta t = 10$ sec., or, $\frac{dQ_p}{dt} = 8.7$. If a surge tank, such as that shown in Fig. 2, had been used, $y = 984.2 - 934.2 = 50$ ft., approximately, and since $h_f = 2.7$ ft.,

$$\frac{dq}{dt} = 320 \frac{32.2}{17\,000} (50 - 2.7) = 28.6$$

The flow in the tunnel, therefore, would increase as fast as the flow in the penstock, and the drawdown would be less than y until the penstock flow begins to increase faster than the tunnel flow can follow.

This point is determined by:

$$\frac{dq_p}{dt} = \frac{dq}{dt} = A_t \frac{g}{L} (y - h_f)$$

As y is known, h_f is determined, and this determines the value of q, beyond which a deficiency between penstock flow and tunnel flow occurs.

APPENDIX III

The general differential equations may be obtained from Equations (4), (10), and (8), by allowing Δt to approach dt. Then, from Equation (4):

$$d y = d h_a + d h_f \ldots\ldots\ldots\

DISCUSSION

F. G. Baum,* M. Am. Soc. C. E. (by letter).—The design of the proper surge tank for power plants has been considered by many engineers. For the conditions as shown by Figs. 2 and 6, the matter was first analyzed for a plant for Peru in 1916, as follows:

1.—For hydrostatic balance and to supply the air required, the stand-pipe may be much smaller than that necessary to give the storage required to furnish the additional water for acceleration.

2.—The storage required for acceleration may be at a lower level than that required for the stand-pipe to balance the hydrostatic pressure.

This resulted in the conclusion that the diameter of the stand-pipe should be small, and the first effect of the plant should be to destroy the stand-pipe head so as to start the acceleration of the water in the flow conduit. Then, there would be required in the storage part of the surge tank, enough water to care for the needs of the plant until the acceleration is complete. This gives a logical, economical solution, and a larger surge tank is not warranted.

The solution adapts itself especially where the slope of the ground at the head of the pressure pipe is quite flat, because the storage tank has only a slightly greater slope than the flow conduit, as the bulk of the weight resulting from the storage water is then near the ground and the stand-pipe proper becomes a light affair.

P. Wahlman,† M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest. Although the author's equations seem to have some merit as applied to special cases, such as those illustrated, there is no equation that represents the behavior of a simple surge tank subjected to a load demand, when friction and action of the governor are taken into account. The degree of accuracy obtainable by any equation depends on the physical conditions encountered, and so much latitude is involved for display of judgment that such equations in the hands of the inexperienced are not only of doubtful value, but may lead to serious errors.

The writer is interested in the fact that the conical shape has been adopted for the Kerckhoff plant in preference to the usual cylindrical form, as he has suggested in an article on surge tanks (published in a European technical journal)‡ that, under certain conditions, such a conical shape would be an improvement on the usual cylindrical form or the inverted cone. He was, however, convinced previously that no form of simple tank could compete economically with a tank of suitable shape designed on the differential principle. This being a demonstrable fact and almost universally acknowledged, it is rather surprising that simple tanks are still built, although a considerable saving of money would result from the application of the differential principle.

In order to demonstrate this in the present instance, the behavior of a small differential surge tank of cylindrical shape, when subjected to the same

* Cons. Engr., Water Power, San Francisco, Calif.

† Hydr. Engr., New York City.

‡ *Teknisk Tidskrift*, Stockholm, Sweden, June 16th, 1915.

program of increasing load, has been computed by the process of arithmetic integration and is shown in Fig. 16.

It is evident that this differential tank is better than the accepted design for this particular test. Other tests may not show such a striking comparison, and it should not be inferred that the writer considers the adopted program of load application, whereby the penstock velocity gradually increases during a period of 70 sec. and then remains constant, as representing a severe duty imposed on the tank. A more usual method of studying these surges is to take into account the action of the governor and to adjust the penstock velocity according to the variable head during a surge so that the power output remains constant. For such requirements as are usually demanded, this little differential tank would be inadequate, but so would the accepted design for the same reason. The surge would ordinarily be considered too unstable and too likely to keep in perpetual motion through a wide range.

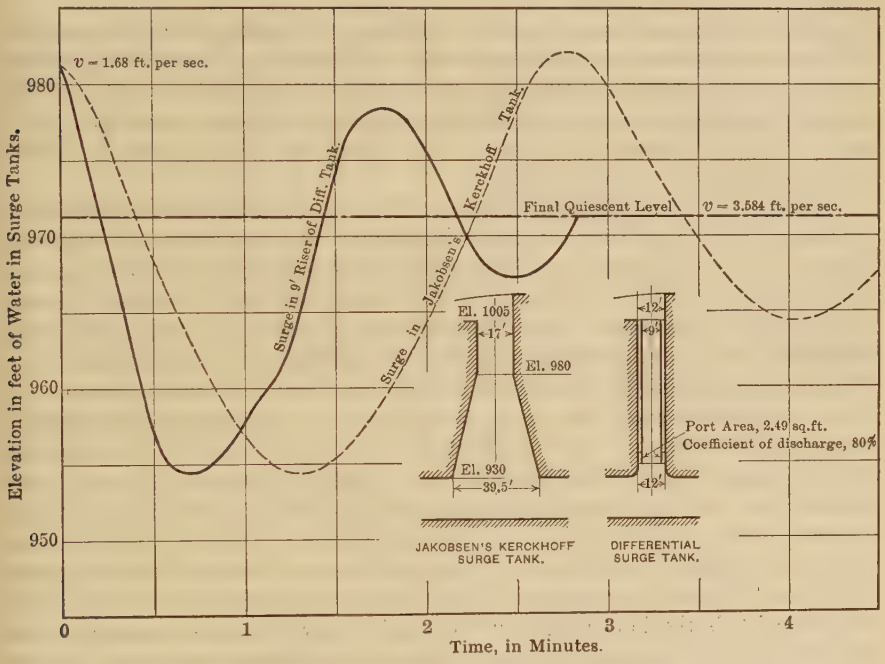


FIG. 16.

In order to obtain this comparison on a precisely uniform basis, the surge in the Kerckhoff design has been recomputed for the same conditions as those assumed for the 12-ft. differental tank of less than one-quarter of the excavated volume. These conditions are as follows: The initial velocity in the tunnel is taken at 1.68 ft. per sec.; the draft velocity in the penstock, expressed in terms of tunnel velocity, is made to vary gradually from 1.68 to 3.584 at the rate of 0.0272 ft. per sec. and thereafter to remain constant at 3.584 ft. per sec.; and the drop in hydraulic gradient is assumed to be 1.027 times the square of the tunnel velocity.

Fig. 16 shows that the surge in the 12-ft. tank is no greater than that in the Kerckhoff design and that it is more favorable in respect to its subsequent liveliness.

The volume of a differential surge tank, which the writer would recommend for this case, if the conditions were of the usually expected severity, would be considerably larger than the one shown, but it would also be much smaller than an equivalent simple tank, regardless of its shape.

CHARLES P. DUNN,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer believes that nearly all surge-tank designs may be placed in the author's Class I, and rarely in his Class II. The author calls attention to this in his Introduction, where he states:

"* * * it also becomes necessary to provide water storage near the plant, so that the load on the plant can be increased as fast as modern operation requires. A surge tank designed for this last condition will also relieve surges * * *".

A tank which has storage capacity sufficient to handle sudden increases of load in a satisfactory manner, may easily be designed so that it will relieve, by spilling, any increases in conduit pressure due to rejection of load, particularly in plants equipped with dependable relief valves of large capacity. Under any ordinary condition of operation, the money lost in the form of spilled water is so little as to be negligible.

The writer agrees with the opinion of the author expressed on page 1365 in regard to the futility of great refinement in surge-tank calculations, because the basic assumptions are affected by the designer's skill in estimating certain future conditions. However, one inaccuracy quite commonly found in surge-tank calculations seems great enough to merit attention. It is the assumption of a straight-line variation in penstock flow to correspond to a straight-line variation in load, and a uniform penstock flow for a uniform load. This introduces an error because, up to the limit of gate capacity, the governors will adjust the gates to use the quantity of water necessary to generate the power demanded, and for a given amount of power the penstock flow will vary inversely as the head (neglecting the effect of slight variation in efficiency). This inaccuracy may cause a slight error in choosing the size of a tank, because the effectiveness of a surge tank is measured by the difference in water level in the tank between normal and the low point of the first down surge. The critical point is the lowest head under which the wheels must operate during the period of adjustment, because if the sudden demand for power reaches a value greater than the capacity of the wheels at the time of low-water elevation in the tank, a disturbance in frequency will result. A calculation of the first down surge, as given in Fig. 9, with the penstock flow corrected for variation in effective head, shows the low point of the first surge to be approximately Elevation 949.2, or 4.9 ft. lower than shown in Fig. 9 (period about 88 sec.).

It seems desirable, in the design of a surge tank as well as in many other features of a power installation, to study the problem from the standpoint of several plants operating interconnected. For example, if several plants of a

* Designing Engr., Skagit River Power, Seattle, Wash.

system have been found by experience to be deficient in their ability to pick up load quickly, such deficiency may be remedied in some measure by providing a generous tank in the next plant to be built. Each individual plant with a long conduit, however, must have surge-tank capacity sufficient that it can operate separately. Two plants with markedly different surge periods should work well together, if the low points of the first down surge are separated by a time interval sufficient that the generating capacity is not at the minimum at the same time in both plants. The period of the surge cannot be controlled at will in a practical design, because it must be subordinated to other more important requirements, but this consideration of relative surge periods is of interest, and, in some cases, may influence the decision as to the size of tank necessary.

In general, those tanks that drop the head quickly to a low value, produce a given regulation of water level with the least volume of tank, which usually means the most economical tank. It is well, however, in studying a design, to balance the cost of any special device for dropping the head quickly, against the cost of the additional volume of tank which would be necessary to effect the same regulation.

The writer believes the shape adopted for the Kerckhoff tank to be the best possible design for a site where the tank is to be excavated in solid rock.

Minton M. Warren,* M. Am. Soc. C. E. (by letter).—Hydraulic engineers are certainly indebted to Mr. Jakobsen for giving them a method of designing surge tanks that is supported by actual operating data on a large tank.

Since the writer's paper on this subject was published, in 1915,† many excellent formulas have been suggested and many methods of computation developed, but a great lack of experimental data on surge tanks still exists.

For a first approximation in surge-tank design, it is perhaps unnecessary to use complicated formulas or tedious step-by-step computations. The simple formulas for surges:

$$S = Q\sqrt{\frac{L}{A_s\, g\, A_t}}$$

and for total time of a complete cycle:

$$T = 2\,\pi\sqrt{\frac{L\,A_s}{g\,A_t}}$$

give at least an idea of the surges and periods, even though they neglect friction and assume that one limit of Q is zero.

Take Mr. Jakobsen's test for load off in which Q decreased from 1 070 to 515 cu. ft. per sec., or $Q = 555$ cu. ft. per sec.; $L = 17\,000$ ft.; $A_s = 225$ sq. ft.; and $A_t = 320$ sq. ft. Using these data in the formula, $S = 47$ ft.: The first upward surge in the experiment was not complete as it spilled over after rising 40 ft., but by continuing the curve (Fig. 11), it is seen that 47 ft. is not far from the truth. The time, T, for a complete cycle by the formula, is 120 sec., which is very close to the time observed in the experiment.

* Eng. Dept., Kalmus, Comstock and Wescott, Inc., Boston, Mass.

† "Penstock and Surge Tank Problems", *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), p. 238.

In the load-on test, a simple formula is not as easy to apply, as the area of the surge tank is varying. For the purpose of supplying load quickly, a tank shaped like the Kerckhoff tank is more effective than a simple one of constant diameter as, first, the head drops more quickly at the start, thus speeding up the water in the tunnel; and, second, the main water storage is in the lower part.

In assuming A_s, therefore, for use in the simple formula given previously, the diameter of the Kerckhoff tank was taken near the bottom of the surge. This at Elevation 960 is about 26 ft. and $A_s = 530$ sq. ft. Q varied from 536 to 1 182 cu. ft. per sec., or $Q = 646$ cu. ft. per sec. By the formula, $T = 185$ sec., which again is close to the observed time for the first cycle. The downward surge, S, however, is calculated to be 36 ft., whereas the actual observed drop was only about 25 ft. In this case, therefore, the simple formula does not give results sufficiently accurate for even approximate design and, in computing a tank of varying cross-section, it is doubtful whether any simple formula would take the place of the author's method, or a similar one. The rapid damping out of the surges in both load-on and load-off tests is significant, as the tank is without any special devices to accomplish this result.

It would be interesting to know whether Mr. Jakobsen has ever observed any harmonic surges in surge tanks. It has been claimed by some engineers that a simple surge or air tank will often be worse than no tank, as the governor, relief valves, etc., will sometimes start surges in step with the period of the system, which will get larger and larger or, at least, continue for a long time. The writer has never observed this effect, but he has observed a tendency to "breathing" in certain systems, even where there was no sudden load change, that is, the pressure in the pipe line rose and fell in harmonic waves with a definite period.

Another matter of interest would be to know whether any differential devices had been considered in the Kerckhoff tank or any other of the author's plants. It has been claimed that a differential device, such as Johnson's surge tank, will cut down the size and expense of a given tank without decreasing the efficiency. In an air tank designed and built by the writer, a device of this kind was used, as it had been found by experiment on existing tanks that their efficiency was increased thereby. It would seem that some such device might have been used to advantage on the Kerckhoff tank.

B. F. Jakobsen,* Assoc. M. Am. Soc. C. E.† (by letter).—When the writer began considering the design of surge tanks for the plant in Peru, referred to by Mr. Baum, and for the plant of the Homestake Mining Company, he was not familiar with the principle of the differential surge tank. Consideration of the problem led to the writer's opinions as given in connection with Figs. 2 and 3. The articles by Mr. R. D. Johnson, on differential surge tanks, quoted in Appendix IV, were found in searching for references on surge tanks. It will be seen that the differential surge tank is a clever combination of Figs. 2 and 3.

* Cons. Hydr. and Elec. Engr., Fresno, Calif.
† Now M. Am. Soc. C. E.

The writer is not acquainted with the article in *Teknisk Tidskrift,* referred to by Mr. Wahlman, and he cannot share his opinion that the differential surge tank is always the more economical. Fig. 16 is interesting, and if load changes such as are there considered were the only ones to be provided for, the differential surge tank would be preferable. In Fig. 16, the load-on period lasts 70 sec., and it will be noted that the head in the Kerckhoff surge tank averages about 10 ft. more than that in the differential surge tank. As the average head on the plant is about 330 ft., this 10-ft. difference in surge-tank head, or about 3% of the head on the plant, is of little importance. With a more rapid change of load, however, this difference may be considerable in that the differential surge tank approaches the surge tank shown in Fig 2.

The disadvantage of Mr. Wahlman's design lies in the fact that it fails in case of sudden load demands, caused by a short-circuit, or by another plant being suddenly disconnected from the system, whereas the Kerckhoff surge tank can carry a great deal of over-load for a short time, until the short-circuit is cleared, or until slower regulating plants can increase their output sufficiently to relieve the Kerckhoff power plant.

Between Elevations 980 and 930, the Kerckhoff surge tank has a volume of 33 900 cu. ft., whereas Mr. Wahlman's differential surge tank has a volume of 5 650 cu. ft., or only one-sixth of the volume of the Kerckhoff tank. At the lowest elevations, this difference approaches $\frac{40^2}{12^2}$, or about 11: 1.

The energy available for the system is evidently the potential energy stored in the surge tank, and this can be released only by lowering the water surface in the tank. As an example, assume water in the surge tank standing at Elevation 980 (which would mean a tunnel velocity of about 1.68 ft. per sec.), and assume a sudden increase of load, corresponding to a tunnel velocity of 3.68 ft. per sec. The area of the tunnel is 320 sq. ft., so that an additional velocity of 2 ft. per sec. for 12 sec. represents 7 700 cu. ft. of water. The differential surge tank contains only 5 650 cu. ft., the difference being supplied by the increased velocity in the tunnel. The average accelerating head during the period is about 25 ft. for the differential surge tank. Under this assumed load change, it takes about 12 sec. to empty the differential surge tank, whereas the Kerckhoff surge tank can supply this draft for 53 sec., even if the water supplied by the increase of the tunnel velocity during this period is neglected, and the average accelerating head is not much less than 25 ft., and may be greater. The writer, therefore, cannot agree that Mr. Wahlman's differential surge tank is comparable with the Kerckhoff surge tank, except for load changes about the same as those assumed in Fig. 16.

The San Joaquin Light and Power Corporation supplies power for a number of induction motor-driven pumping plants. When, owing to a short-circuit or other system disturbance, the voltage or the frequency of the system, or both, drop much below normal, these pumping plants are likely to be "tripped off" when the disturbance is cleared, and the voltage and frequency quickly return to normal. As most of these pumps have to be started by hand, this means that the farmer must leave his work, causing unfavorable comments

and dissatisfaction with the service. The Kerckhoff plant was originally equipped with Tirrell voltage regulators, but the exciter turbines were not equipped with governors, because it was not thought to be necessary. It was found, however, that on short-circuits, the exciters would slow down and the plant was unable to maintain its voltage. Therefore, governors were installed and the operation of the plant has since been satisfactory, according to Mr. E. A. Quinn, the General Superintendent. The ability, therefore, of the surge tank to sustain a short-circuit is of importance. This matter should have been presented more clearly and the writer is glad an opportunity has been afforded to discuss it.

Mr. Dunn is right in calling attention to the influence of the drop in the surge tank on the flow in the penstock or on the power delivered to the wheels. In the case considered, this involves an additional drawdown of only 5 ft., according to Mr. Dunn, but it is not safe to assume that this is always negligible. This fact makes the surge tank shown in Fig. 2 undesirable with medium-head plants, and it also applies, in a measure, to the differential surge tank.

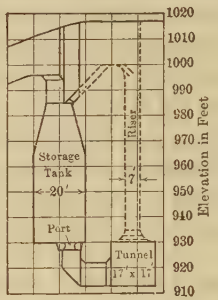


FIG. 17.

Referring to Mr. Warren's discussion, the writer experimented with the simple formulas which he gives (that is, sine curve variation), but found that the variable area of the surge tank cannot be estimated beforehand. Nevertheless, the formulas may be used, especially with those given in Appendix II, for the surge tank shown in Fig. 2, in order to obtain some idea of the desirable size of the surge tank.

The writer has not observed any harmonic surges; such surges are possible, but it is believed they are improbable. No such surges have been observed in the Kerckhoff plant. This plant has been subjected to heavy short-circuits on the 110 000-volt transmission lines, followed by the opening of the line switches, which reduced the plant load instantly from full load to no load, and no difficulties have been observed.

The Larner-Johnson Valve and Engineering Company was asked to furnish a design of a differential surge tank and submitted, accordingly, the one shown in Fig. 17, consisting of two independent shafts, one 7 ft. in diameter rising directly above the tunnel, and the other 20 ft. in diameter offset from the tunnel and connected at the bottom to the tunnel and at the top to the riser. This tank would be as expensive to excavate as the Kerckhoff surge tank, and it is not as desirable for the following reasons.

This differential surge tank would permit an increase from no load to full load in 3 min., as will also the Kerckhoff surge tank. For such a load increase, this tank is comparable with the Kerckhoff tank. However, when the load is increased instantaneously, or within a few seconds, from 1 000 to 1 500 sec-ft. (1 500* sec-ft. being about full load on the plant), the water surface in the differential surge tank is decreased from Elevation 965 to Eleva-

* The following computations are based on a tunnel 17 by 17 ft. and Kutter's $n = 0.040$.

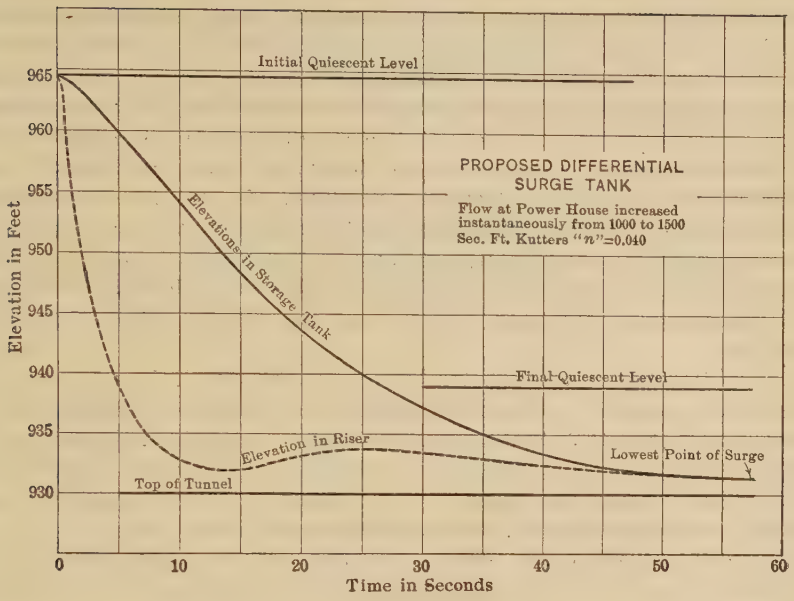
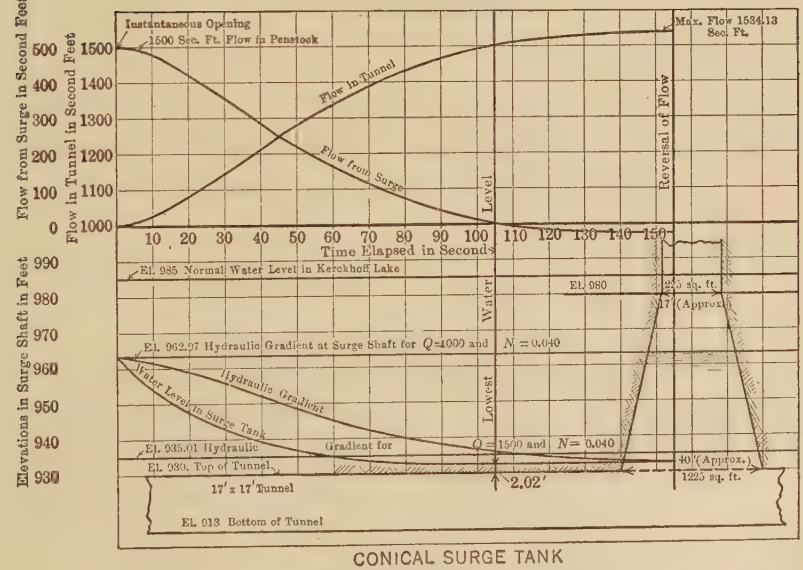


FIG. 18.


Instantaneous Opening
1500 Sec. Ft. Flow in Penstock
Max. Flow 1534.13 Sec. Ft.
Flow in Tunnel
Flow from Surge
Reversal of Flow
Level
Flow from Surge in Second Feet
Flow in Tunnel in Second Feet
Time Elapsed in Seconds
El. 985 Normal Water Level in Kerckhoff Lake
El. 980
(Approx.)
El. 962.97 Hydraulic Gradient at Surge Shaft for Q=1000 and N=0.040
Water Level in Surge Tank
Hydraulic Gradient
Lowest Water
El. 935.01 Hydraulic Gradient for Q=1500 and N=0.040
El. 930. Top of Tunnel
17' x 17' Tunnel
2.02'
40' (Approx.)
1225 sq. ft.
El. 913 Bottom of Tunnel
Elevations in Surge Shaft in Feet

CONICAL SURGE TANK
INSTANTANEOUS OPENING FROM 1000 TO 1500 SECOND FEET
5 Sec. Time Intervals from 0 to 65 Seconds
10 Sec. Time Intervals from 65 to 155 Seconds

FIG. 19.

tion 932, or the head on the plant is reduced by 33 ft. (about 10% of the total head) in 10 sec.; whereas, in the Kerckhoff surge tank, the decrease in head is only 8 ft. in 10 sec. (Figs. 18 and 19). Both surge tanks meet the condition imposed on them, but during the first minute after the turbine gates have opened, the plant with the Kerckhoff surge tank supplies, on an average, nearly 2 000 kw., more than the one with the differential surge tank, and 10 sec. after the moment of regulation, the Kerckhoff surge tank supplies about 2 700 kw. more than the differential surge tank. As this is nearly the ultimate capacity of the plant and for nearly full-gate and at a time when every kilowatt counts, it is an advantage in favor of the Kerckhoff surge tank. The normal full load on the Kerckhoff plant is 35 000 kw., although 40 000 kw.* have been generated for several hours, when the tail-race elevation was relatively low.

The differential surge tank has its advantages, but it is not always the best possible type, as is shown by the previous discussion. Each plant requires a special study, and the size and shape of the tank depend on the conditions to be met and the physical constants of the plant. The success of the design depends also largely on the designer's ability to appreciate properly the operating conditions which will be imposed on the plant.

In conclusion, the writer wishes to thank those who have participated in the discussion, as well as Mr. E. A. Quinn for his co-operation in conducting the tests.

* Or about 1 750 sec-ft.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

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Paper No. 1505

FLOOD PROBLEMS*

A SYMPOSIUM

By Messrs. J. G. Sullivan, Gerard H. Matthes, Nathan C. Grover, John R. Freeman, J. A. Ockerson, Roy N. Towl, Arthur P. Davis, C. E. Grunsky, and Charles H. Paul.

With Discussion by Messrs. Morris Knowles, Harrison P. Eddy, J. Albert Holmes, D. W. Mead, Arthur O. Ridgway, J. B. Challies, George M. Lehman, W. H. Breithaupt, Adolph F. Meyer, John Millis, C. McD. Townsend, Thomas H. Wiggin, and E. W. Lane.

* Presented at the meeting of the Society at Dayton, Ohio, on April 5th, 1922.

FLOOD CONDITIONS IN CANADA

By J. G. Sullivan,* M. Am. Soc. C. E.

About 1919, the speaker became associated with the Manitoba Drainage Commission and, possibly, on account of this association, the Government of Manitoba has asked his advice on some real "flood problems". The two important points on which the Commission desires confirmation of its conclusions are:

First.—Does the clearing of timber from land and the cultivation of the land increase the run-off?

Second.—If the first question is answered in the affirmative, what responsibility, if any, attaches to the lands from which the extra water comes; in other words, what lands can be included in an Injuring Liability Clause of the statutes?

The Manitoba Drainage Commission has reported to the Provincial Government on this point, as follows:

"Our final recommendation in the matter of boundaries is the extension of the boundaries of any drainage district to include all lands whose surplus waters drain into said district and are carried by any artificial channel through it to a natural outlet. This is a natural division of lands, any other division leads to disputes.

"We do not contemplate that all lands within the new boundaries should be taxed. In the case of Drainage District Nos. 19 and 8, the lands west of these districts on the Riding Mountain are in a Government reservation and covered with timber, which latter fact, in any case, would relieve them of taxation for drainage cost."

In order to explain conditions and circumstances relative to a part of the valley of the Red River, south of Winnipeg, the speaker will quote further from the report of the Drainage Commission, as follows:

"In Drainage District No. 2, aside from the claim of inequitable taxation, complaints have come from three classes: First, from those living in the low lands which never had been drained, although the land has been taxed the same amount as all the other lands in the district. Second, from those who had at one time drains that became ineffective through lack of maintenance, and these people pointed with scorn to what they termed 'Government drains' which were full of soil which had blown into the ditches or had become so foul with uncleared brush, and other growths, that they could not properly function. It was surprising how few people realized that the municipalities and not the Government were the parties responsible for maintenance work. The complaint of those who never had protection is well founded, and they should be given relief as soon as it is possible to finance the necessary funds. The third class of landowners to complain are those suffering from flood waters, which waters spread over the low lands and destroy crops as occurred the first part of July in this year, or they occur in the spring and leave the land so wet that crops are not put in, or else put in so late that they are apt to suffer from rust or frost.

"Your Commission is of the opinion that the losses in Drainage District No. 2 for the past ten years from this flooding would be at least ten times as great as the cost of works necessary to protect them from such floods.

* Cons. Engr., Winnipeg, Man., Canada.

"As to the cause of this trouble, part of it is due to lack of maintenance of ditches, which work according to statutes should be attended to by the officers of the various municipalities, but the Government cannot lay all the blame on the municipalities for the reason that they had the benefit of the advice of experts who realized the result of not properly looking after the ditches, and they should have realized that the Act relative to maintenance work was not practicable, nor would its enforcement result in equitable distribution of cost, and the Government should have changed same, or failing this, should have made it their business to force the municipalities to keep the works in proper condition regardless of results. However, the greatest factor causing damage from floods is the changed conditions since the districts were first formed.

"When Drainage District No. 2 was first formed, comprising 441 017 acres, there was nearly as much land west of the district only partly cultivated and quite extensively covered with timber. In 1887, in seven townships, in Ranges 4 to 6, there were 4 177 acres of cultivated land; in these same townships in 1890, the cultivated land was 7 994 acres; in 1915 the cultivated land amounted to 82 000 acres; and now practically all the land between the foothills and Drainage District No. 2 is cleared of timber and under cultivation, about 600 000 acres. Nearly as much more land is on the higher ground in this watershed and at the time Drainage District No. 2 was formed, nearly all the latter area was covered with timber. Since that time approximately 75% of these high lands have been cleared and put under cultivation, and the lands between the foothills and Drainage District No. 2 have been artificially drained by ditches built with Government aid and by road ditches. All of the work mentioned results in hastening run-off and, therefore, increasing run-off, for anything that hastens the run-off increases the same by decreasing absorption and evaporation. The result is that this extra water comes rushing down on the lower lands in Drainage District No. 2 at a time when the main channels are full of snow and ice, with the result that large areas are flooded and millions of dollars lost annually by failure to get a crop in or by rust on account of the lateness of getting in the crop. No matter how well the original channels were designed and no matter how well they were maintained, they would not now be capable of properly taking care of the extra rush of waters that come from the higher grounds on account of the changed conditions, and your Commission believes that the lands from which this water comes should help to pay a portion of the cost of carrying these waters to the Red River. It thinks that the proportion should be small compared with what should be paid by land that without drainage would be useless."

In a hearing before a Legislative Committee on this subject, contradictory testimony was given by old-time farmers. The men from the higher lands testified that the run-off from cultivated lands was much less than that from wild or timbered lands and stated, further, that the floods were much greater 30 or 40 years ago than at the present time; whereas, the witnesses from the low lands testified to opposite conditions. The Commission and its engineers contended that, in general, the run-off from a cultivated field in the soil of this country is greater from a summer fallow field than from one of wild land, or from timbered land. Further, in the case of melting snow under a warm sun or warm rains, when there is frost in the ground to a depth of 6 to 8 ft., the run-off from the bare fields is much faster and is, therefore, greater than from the wild lands covered with heavy grass or timber. In such cases, with the water coming down in a rush when the ditches are full of snow and ice, the danger from floods is an annual menace to the lower lands. This argument has been borne out by experience in this District during the past 15 or 20 years, the spring flooding becoming more excessive and occurring more

regularly as the timber has been cleared and the land placed under cultivation. It is recognized that there are exceptions to this general rule; one exception would be when the ground was covered with, say, 2 ft. or more of snow, with the temperature during the day not more than 40°, and with frost at night. Should this condition prevail for 2 or 3 weeks, all the snow would disappear through evaporation, and with a high barometer the moisture might be precipitated outside the water-shed and the danger of a flood would be less than if the land had been covered with timber and the snow held back until continuous warm weather and warm rains should come. If the weather turns from cold to continuous high temperatures, with the ground covered with snow, then, as is stated in the report, the timber would hold back the snow until the high temperatures had melted the snow and ice in the ditches. As to the claim that timbered lands in this District increase the danger of excessive floods, it will have to be admitted that there are conditions under which this is true, as has been indicated.

As a railroad man, the speaker has had several years' experience on the North Pacific Coast, and any winter that the snowfall appeared to be a little greater than usual, the operating officers began to worry about danger from spring floods. After long experience, the speaker came to the conclusion that there was enough snow in the mountains every year to cause disastrous floods if conditions were right, namely, a continuous warm spell of weather with or without rain. In early spring, this condition is almost impossible, therefore, the time to begin to worry is when there is a cold late spring. Such were the conditions in the Northwest in 1894. The weather remained cold until May 24th, when it suddenly turned warm and, on June 9th, the water in Kootenay Lake was 31 ft. above low-water mark. The water was 3 or 4 ft. deep in the Canadian Pacific Railroad Station at Nelson, B. C., Canada. On the Arrow Lakes, the water rose to about the same height.

This brings up the flood problem of Manitoba. Winnipeg is in what was the bottom of a lake not so long ago, as geologists count time, and should not have been located there. The Red River draining this area has not cut a sufficient channel to prevent dangerous floods. Another adverse feature is that this river flows north. The present problem is to determine what will be the effect on the city and the lands between the International Boundary and St. Andrew's Locks, in the valley of the Red River, of works of flood control (other than the construction of reservoirs by the people of Minnesota and the Dakotas). Where natural reservoirs are not available, the usual practice is to increase the carrying capacity of the river by enlarging, straightening, and diking. It is possible to decrease the Red River to one-half its present length. The distance from Emerson to Winnipeg, *via* rail, is 60 miles. The river survey shows a distance of 120 miles and the speaker's information is that the same ratio obtained south of the Boundary. The speaker has advised the Government of Manitoba, that, in his opinion, the work being done within the Province, of draining land and preventing local flooding in the tributaries of the Red River by double diking, etc., combined with that which is being done and may be extended south of the Boundary, in time will bring about a condition whereby the City of Winnipeg and other property between

the Boundary and St. Andrew's Locks, will be in danger of damage from floods annually, and, further, it is the frequent damage that is to be feared. For instance, with land damaged by floods annually to the extent of $1 per acre, with money at 5%, one would be justified in spending nearly $20 per acre to prevent such damage; but if the land is damaged to the extent of $10 per acre, by floods once in 30 years, one would not be justified in spending more than $3 per acre to prevent this damage.

To sum up, what the speaker wants to know, is, does clearing land, building drains, and diking streams, tend to increase the frequency of floods and, if so, is one justified in assuming that there is a responsibility on account of "injuring liability" attached to the higher lands?

FLOODS ON SMALL STREAMS CAUSED BY RAINFALL OF THE CLOUDBURST TYPE

By Gerard H. Matthes,* M. Am. Soc. C. E.

The object of this paper is to focus attention on the subject of floods caused by localized rains of great intensity, often styled cloudbursts, which affect principally the smaller streams. As far as the speaker is aware, no comprehensive study of such floods has appeared in print. The difficulties in the way of collecting reliable data on this subject are too well known to require special emphasis. These difficulties rather than the infrequency of cloudburst floods account for the paucity of available data and the incomplete state of knowledge concerning this not only common but most destructive type of run-off. The speaker believes that much valuable information hitherto unpublished is contained in the files of engineers throughout the United States, and that these data, if brought together, collated, and analyzed by a competent agency, would go far toward throwing much needed light on this subject.

Definitions.—The term "small streams", is applied herein to any watercourse in which the maximum rates of flood flow are caused, not by prolonged and widespread heavy rains, as in the case of rivers, but by downpours of exceptional intensity, of short duration, and covering areas rarely exceeding 50 sq. miles. Unscientific as the term, "cloudburst", may be, it is a common designation for such rains and has been used freely by engineers. It has been adopted by the speaker for want of a better appellation. The term, "excessive precipitation", in use by the U. S. Weather Bureau, is not satisfactory for the purposes of this paper, in that it is too general in its definition, and not sufficiently expressive of the particular kind of excessive precipitation under consideration. The speaker has been inclined to the use of the term, "torrential rain", but has found the one word, "cloudburst", simpler for his purpose. In further defense of its use, it may be of interest to state that other languages besides the English, notably the German, have literal equivalents for this term. For the sake of brevity, the term, "cloudburst flood", has been used by the speaker to describe floods in small streams, caused by cloudbursts or by precipitation approximating that of cloudbursts. No limit is specified as to the size of drainage areas of small streams as defined herein, as the definition given places an automatic limit which must vary with local physiographical and meteorological conditions.

Distribution and Physiography.—Cloudbursts appear to be of common occurrence throughout the entire United States south of the 42d Parallel, some localities being more subject to them than others. In northern latitudes, they are not so common, owing, evidently, to the longer winter seasons during which strong convection currents are rare and the atmosphere is not capable of holding large quantities of moisture.

Topography is one of the governing factors in cloudburst formation, but the relation appears to be little understood. There is a feeling among some Western engineers that cloudbursts are peculiar to the arid regions. This is

* U. S. Asst. Engr., Chattanooga, Tenn.

not borne out by the facts; it is true, however, that in regions of small annual precipitation, like the Western States, cloudbursts form an important part of the annual rainfall, and exert a marked influence on the erosion features of the country. Whether cloudbursts can sculpture the relief of a region so as to make it thereby more susceptible, in turn, to the formation of cloudbursts, is a matter well worthy of investigation. The steep walled arroyos and smaller canyons of the West, no doubt, owe their formation and maintenance largely to this form of run-off. The deeply dissected rims and side canyons of the Grand Canyon of the Colorado River, a region noted for cloudburst precipitation, strongly suggest an intimate relation between topography and cloudbursts there.

Plains and table-lands seem to have a bearing, presumably, in promoting atmospheric movements which are important in bringing, to the scene, moisture from over a vast radius. The entire east front of the Rocky Mountains in Colorado, especially along the foot-hills, has long been known as a zone in which cloudbursts are of common annual occurrence. The spectacular storms which took place there in June, 1921, except for their unusual severity, present no novel features in that respect. At the higher elevations in these mountains, cloudbursts are comparatively rare.

It is claimed that certain localities, because of peculiar topographic conditions, are immune from cloudbursts. The City of Chattanooga, Tenn., situated in a wide valley surrounded by high hills, enjoys this reputation. Less than 50 miles to the south, in Georgia, cloudbursts annually inflict much damage.

Whether proximity to bodies of water, such as large rivers, lakes, or the ocean, promotes the occurrence of such storms, is questionable. The records of Erie, Pa., and Baltimore, Md., given subsequently, seem to suggest that the nearness of lake or ocean has a bearing on the frequency of cloudbursts. However, reference to the record of Cherry Creek, at Denver, Colo., would dispel any such idea.

The bearing which physiographic features have on this entire subject, is so important as to invite the most detailed study. It is evident that a knowledge of both frequency and maximum rates of rainfall, that is, the class of information needed by the Engineering Profession in designing structures on smaller streams, hinges on a proper understanding of topographical and meteorological conditions as affecting cloudburst occurence. If any study has been given to the former, it has not come to the speaker's notice.

Damage.—Unlike river floods which occur principally in the winter and early spring and are often beneficial by enriching the soil with silt deposits, cloudburst floods occur nearly always during the crop-growing season, and, next to tornadoes, are the most destructive agency with which the farmer has to contend. Washouts of railroad and highway embankments, destruction of bridges, culverts, and buildings, and loss of life, are common forms of cloudburst damage which amounts to millions of dollars annually in the United States. As in other floods, the invisible damage often is considerable. The detouring of trains over neighboring lines, which, in many cases, costs more than the physical damage suffered by a railroad; interruption and loss of busi-

ness of all kinds; and, finally, the cost of human life, are some of the items commonly omitted in estimating flood damage. Many cloudburst floods are unnoticed by the press and by Government or State Bureaus. In fact, it appears to be nobody's business to study cloudburst occurrences. The speaker believes that if the toll taken annually by these floods were better known, it would attract greater attention to the problem of providing against them, and he takes this opportunity to emphasize the importance of collecting reliable statistics on this subject. This should be made the duty of some competent State or Federal agency.

Effect on Parent Stream.—Little damage usually results along large rivers from the sudden emptying into them of one or even two tributaries carrying cloudburst floods, unless the main stream itself should happen to be at a high stage, in which case serious overflows may result. Ordinarily, however, small streams, overtaxed by cloudburst run-off, cause only minor rises in the larger streams to which they are tributary. Perhaps the most harmful effect in such cases is the deposition of a vast quantity of débris in the channel of the larger stream, at a time when its waters are not capable of transporting such débris. The speaker has seen such accumulations of débris spread out fan-shaped at the mouth of the offending tributary, resembling, on a small scale, the well known alluvial cones of the California mountain streams where they emerge from the Sierras. A noteworthy feature of such débris fans is their symmetry, indicating that the flow in an up-stream direction in the main river channel was, temporarily at least, nearly as strong as that moving down stream at the point of confluence. The speaker has been unable to ascertain the time required for a large stream to carry off such accumulations, but this, obviously, would depend on the frequency and size of its floods. This emphasizes the economic importance of floods in maintaining a proper balance in Nature's household. A river system, in many respects, is like an extensive sewer system—periodical flushings are essential to the proper maintenance of its channels. The removal of débris resulting from natural erosion agencies, such as cloudbursts, can be accomplished only through flood flow.

Rainfall.—No attempt will be made to discuss specific rates of cloudburst rainfall. Such data as have been obtained by the U. S. Signal Service, the Weather Bureau, and other sources, for the most part, are quite accessible in printed form. Such figures are of general interest only, and convey little that is of direct value to this discussion, lacking as they do, in the majority of cases, information regarding the distribution of the precipitation both as to time and areas covered. The spasmodic surging nature of cloudburst rainfall, accompanied by exceedingly localized concentration at a few points, renders ordinary rain-gauge observations of doubtful utility. Only self-registering instruments can be of use in such cases, and such instruments are so few in number as to furnish little information from which to draw conclusions. The recording mechanism of these instruments is not well adapted for clearly registering the tipping of the receiving bucket at each 0.01 in. of rain during cloudburst intensities. In one case, where 0.48 in. of rain fell in 8 min., the chart was so crowded that the ink lines had run together, and it was only by measurements of the water in the lower container that

the number of times the bucket had tipped could be ascertained. The chance of obtaining an accurate record of cloudburst rainfall by one rain gauge located within the area of heavy precipitation, belongs, thus far, in the category of fortuitous events. An instance where a fair record of this kind was obtained, is the storm of July, 1914, near Cambridge, Ohio, when 7.09 in. of rain fell in 90 min.*

Field observations made by the speaker, immediately following cloudbursts, have convinced him that such phenomena are often characterized by concentrated local falls of water lasting presumably only a few minutes and recurring probably at intervals at different points within the area of maximum rainfall. This is evidenced by surface indications in localities not subject to erosion by flow in natural watercourses or overflow from the latter. Thus, immediately after the cloudburst of July 10th, 1914, near Scranton, Pa., the speaker found scars high on a mountain side, that were from 3 to 5 yd. wide and nearly 10 yd. deep. Many cubic yards of earth had been torn as if by a jet from an hydraulic giant. Similar scars, but of smaller dimensions, have been noted in Ute Pass, Colo., and at Dotsero, Colo., on other occasions. In these cases, there was no collecting area immediately above these scars, that could have produced sufficient concentration of flow on the ground surface to effect the removal of so much earth, and all the indications pointed to the action of water descending directly from the clouds.

Instances of cloudbursts within towns or cities corroborate this phase of cloudburst precipitation. A remarkable case of this kind occurred at Pittsburgh, Pa., on July 26th, 1874, at 6:30 P. M., when a cloudburst described as having formed by the coming together of two dark clouds, broke over the section of the city known as Allegheny. In a few minutes, more than 100 human beings had been swept down Madison Avenue into the Allegheny River, and 70 were reported to have drowned. About 50 houses were wrecked. In the same storm, cloudbursts occurred at Wood's Run, where 15 people were drowned, and at Sawmill Run, where 25 met death. These two small streams are in the immediate neighborhood of Pittsburgh. Nothing is known in regard to the rainfall during these occurrences.

The question may well be raised whether the study of cloudburst floods can be profitably undertaken through study of rainfall records as obtained by the present system of observation. Records showing that 6 or even 10 in. of rain fell within 1 hour are of small value when accompanied by the further statement that most of it fell in a part of that time. Statements of this kind are only too common. The average observer at a co-operative station cannot be relied on to make detailed observations at such times.

Run-Off.—Engineers are primarily concerned with the rates of maximum run-off and it is fortunate, therefore, that in cloudburst floods such rates are more readily ascertainable than the rates of precipitation which give rise to them. Even so, the information available is meager, and does not justify the drawing of definite conclusions at this time.

* For a description of this storm and a map showing isohyetals, see page 16 of the "Ohio Water Problem", by C. E. Sherman, M. Am. Soc. C. E., in Ohio State University *Bulletin No. 10* (Vol. XX).

Being essentially of the nature of heavy thunder-storms, cloudbursts frequently occur during or in conjunction with them; they cover only small areas within the main storm, and have been known to recur again and again as the thunder-storm progresses. However, they are often isolated, and in such form they offer the best opportunity for detailed study. The storm at Long Level, Pa., described herein, was of this type. An outstanding feature of the isolated form is the comparatively small quantity of total run-off notwithstanding the extraordinary peak rates of flow. This is well brought out in descriptions of the floods of June, 1921, at Pueblo, Colo.,* and of September, 1921, at San Antonio, Tex.,† and afford the comforting thought that flood-control measures, especially by the retarding-basin method, may be economically possible in such cases.

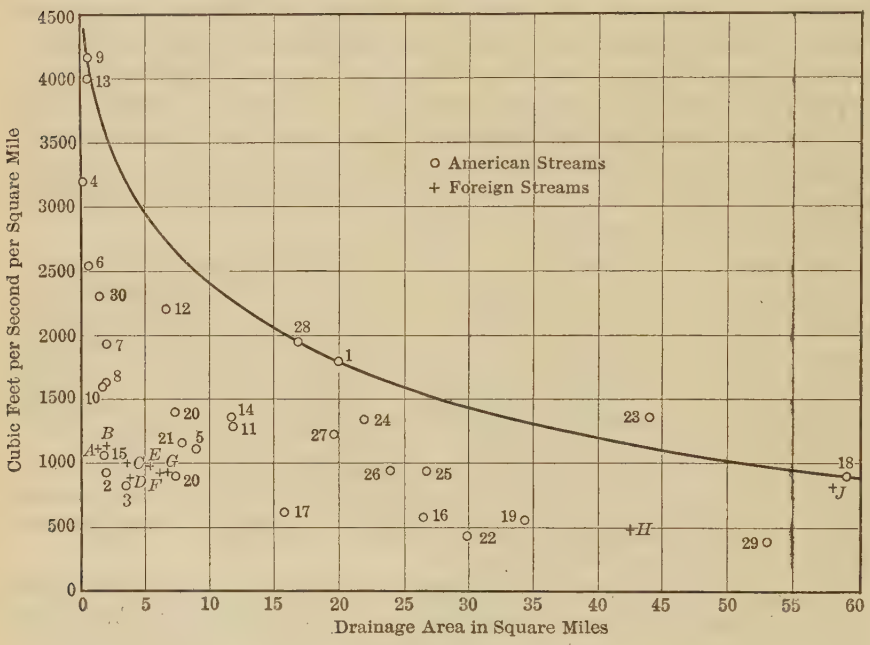


FIG. 1.

Fig. 1 shows a curve of cloudburst run-off compiled from data on American and foreign streams, and Table 1 gives data pertaining to the cloudburst floods. The data derived from larger drainage areas are often complicated by the records of lesser rates of rainfall over parts of those areas, and as they do not offer so clean-cut a comparison, they have not been utilized. Furthermore, there is reason to believe that, for small drainage areas, high rates of run-off offer more nearly comparable results for different sections of the United States.

* See p. 1.
† See p. 354.

TABLE 1—CLOUDBURST FLOOD RECORDS.

	Stream.	Locality.	NORTH AMERICAN STREAMS: Date.	Drainage area, in square miles.	Peak run-off, in second-feet per square mile.	References.
1	Willow Creek	Heppner, Ore.	June 14, 1903	20	1800	*Water Supply Paper No. 96*, U.S.G.S.
2	Cherryvale Creek	Cherryvale, Kans.	.....	2.0	930	*Transactions*, Am. Soc. C. E., Vol. LXXVII.
3	Estanzuela River	Monterey, Mexico	Aug. 25, 1909	3.5	825	*Engineering News*, Sept. 23, 1909.
4	Beacon Brook	Fishkill, N. Y.	July 14, 1897	0.25	3 200	New York State Engr.'s Rept., 1902.
5	Arroyo	Indiole, N. Mex.	July 19, 1915	8.87	1 105	*Bulletin*, Am. Ry. Eng. Assoc., August, 1915.
6	Mann's Run	Creswell Station, Pa.	July 15, 1914	0.67	2 540	Water Supply Comm. of Pa.
7	Indian Run	Letort, Pa.	" " "	2.1	1 930	" " " " "
8	Canadochly Creek	East Prospect, Pa.	" " "	2.2	1 630	" " " " "
9	Bulls Run	Long Level, Pa.	" " "	0.58	4 170	" " " " "
10	Green Branch	Bridgeville, Pa.	" " "	1.7	1 595	" " " " "
11	East Fork, Honey Creek	New Carlisle, Pa.	July 29, 1918	11.8	1 285	Miami Conservancy District.
12	East Fork, Honey Creek	New Carlisle, Pa.	" " "	6.7	2 210	" " "
13	Dookers Hollow	North Braddock, Pa.	June 10, 1917	0.6	4 000	Water Supply Comm. of Pa.
14	Blue Ribbon Creek	Arkansas River drainage near Pueblo, Colo.	June 3, 1921	6.7	1 360	See p. 64.
15	Arroyo		" " "	1.8	1 060	" "
16	Boggs Creek		" " "	26.5	582	" "
17	Arroyo		" " "	15.8	619	" "
18	Rock Creek		" " "	59	913	" "
19	Peck Creek		" " "	34.4	564	" "
20	Cameron Arroyo		" " "	7.3	1 900	" "
21	Osteen Arroyo		" " "	7.8	1 160	" "
22	Pinal Creek	Globe, Ariz.	Aug. 17, 1904	30	440	*Water Supply Paper No. 147*, U.S.G.S.
23	Elkhorn Creek	Keystone, W. Va.	June 22, 1901	44	1 363	*Engineering News*, Vol. 2, (1902) p. 104.
24	Cane Creek	Bakersville, N. C.	May 20, 1901	22	1 341	" " " " "
25	Olmos Creek	San Antonio, Tex.	Sept. 9, 1921	26.8	930	See p. 368.
26	Apache Creek	" " "	" " "	24.0	947	" "
27	Martinez Creek	" " "	" " "	19.6	1 223	" "
28	Alazan Creek	" " "	" " "	16.9	1 950	See p. 365.
29	Santa Ysabel Creek	Mesa Grande, Calif.	Jan. 27, 1916	53	395	*Water Supply Paper No. 426*, U.S.G.S.
30	Mad Creek	Le Roy, N. Y.	June, 1916	1.5	2 300	*Engineering Record*, June 24, 1916.
	FOREIGN STREAMS					
A	Wittgendorfbach	Germany	1887	1.3	1 115	*Transactions*, Am. Soc. C. E., Vol. LXXVII.
B	Dittelsdorfwasser	"	1887	2.0	1 144	" " "
C	Wittgendorfbach	"	1887	3.6	1 010	" " "
D	Landwasser	"	1887	3.8	895	" " "
E	Kemlitz	"	....	5.4	980	" " "
F	Landwasser	"	1887	6.1	830	" " "
G	Spree	"	1887	6.7	930	" " "
H	Zacken	"	....	42.5	490	" " "
J	Pliessnitz	"	1890	58.0	820	" " "

Cloudburst at Long Level, Pa.—A description of this occurrence is given as being representative of the isolated type of cloudburst. It took place on July 15th, 1914, along the Susquehanna River, in York County, Pennsylvania (Fig. 2), and covered an area of about 30 sq. miles. Field measurements were made on July 18th, 1914, by Howard T. Critchlow, Assoc. M. Am. Soc. C. E., assisted by Mr. W. A. Bowen, under the auspices of the Water Supply Commission of Pennsylvania, the speaker at that time being Division Engineer for the Commission, in charge of flood investigations. Through the courtesy of the Commission these data are here made available for publication. As the field work was done with painstaking care, at a time when traces of the floods

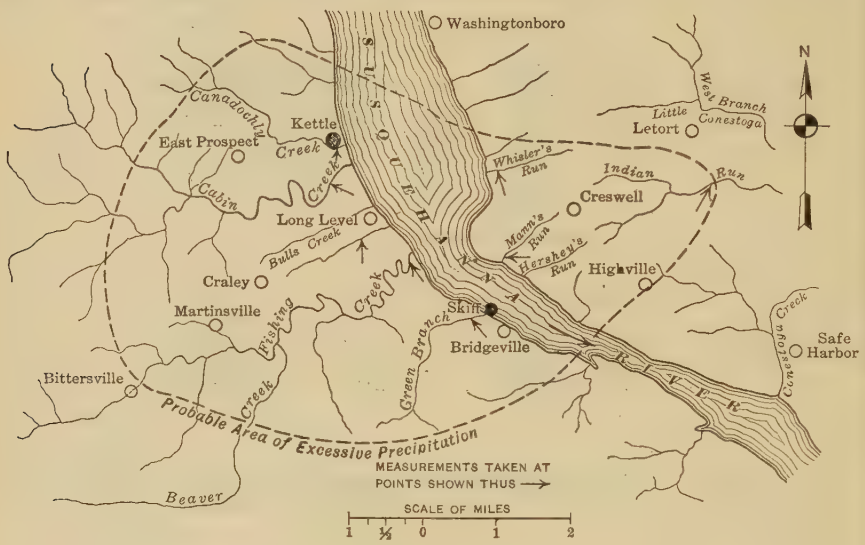


FIG. 2.

were still plainly visible, the speaker has considerable faith in the reliability of the results. The following is quoted from Mr. Critchlow's report:

"The storm broke between 3:30 and 4:00 P. M., of the 15th, and it rained very hard for about 2 hours on the area west of the river, while the precipitation lasted about 1 hour on the east side, beginning about 4:30 P. M. The inhabitants interviewed stated that the downpour was the heaviest they had ever seen, the clouds coming from several directions and hanging over them. There was only slight wind and electrical disturbance. Data obtained from a party near the mouth of Canadochly Creek, consisting of depth of catch in a large iron kettle standing in the open, gave a precipitation of 6.2 in. in 2 hours. Also, data obtained from a party at Bridgeville, consisting of catch in a 16-ft. skiff anchored on west shore of Susquehanna River, gave a precipitation of 5.7 in. in 1½ hours. These data check closely, and the former is especially good. Some hail was reported at Canadochly Creek and at Long Level."

* * * * * * * * * * *

"The data collected on this storm were obtained by W. A. Bowen and the writer as follows: East Side of Susquehanna River, Manns Run, Indian Run, and Whisler's Run on July 18th; West Side of Susquehanna River, Canadochly Creek, Cabin Creek, Cook's Creek, Bulls Run, Fishing Creek, and

Green Branch on July 21st and 22d. All elevations of high-water marks and low water, as well as cross-sections, were taken with small engineer's level, distances being measured with metallic tape. Courses were selected that were not subject to back-water, straight, and of fairly uniform cross-sections and slopes. These conditions were not always to be found, but it is believed that use was made of the best available. Measurements of span and clearance of culverts and bridges visited were made and recorded, but more time and attention was given to the collection of data on rainfall and for computing maximum rates of run-off. The duration of excessive rate of run-off is not definitely known. The small streams seemed to reach their crests within half an hour after the storm began, but the length of time during which flood conditions obtained and rate of fall is an indeterminate factor. They subsided rapidly after the storm ceased. Fishing Creek and Cabin Creek kept at crest for about an hour, the latter rising to crest from 4:00 to 6:00 P. M., remaining at crest until 7:00 P. M., and falling gradually to midnight. The rain here started at 3:00 P. M., and it rained hard for two hours. Any estimate of the total run-off would be subject to considerable error on account of lack of knowledge of rate of subsidence of flood. Only the maximum discharge has been computed, since it is the most important factor in considering flood conditions and the effect on bridges and dams."

TABLE 2.

	Stream.	Drainage area at section, in square miles.	Topography of basin.	Area of cross-section, in square feet.	Hydraulic radius, R, in feet.	Slope, S, ratio.	Value of n, Kutter's formula.	Maximum discharge, in cubic feet per second, per square mile.	Equivalent run-off, in inches per hour.	Remarks.
1	Manns Run....	0.67	Steep, rocky slopes.....	127	2.68	0.0355	0.040	2 540	3.94	100-ft. course, fair conditions.
2	Indian Run (tributary to Conestoga Creek).....	2.06	Rolling, agricultural land.....	378	2.90	0.015	0.035	1 930	3.00	200-ft. course, good conditions.
3	Canadochly Creek.....	3.24	Hilly agricultural land.....	316	4.06	0.013	0.035	1 120	1.73	200-ft. course, good conditions.
4	Cabin Creek...	13.9	Hilly agricultural land.....	255	2.69	0.010	0.035	359	0.56	Computed at timber dam, 70-ft. crest, head = 5.0 ft., by $Q = 3.7\ L\ H^{3/2}$, and by open-channel method for area over right bank.
5	Bulls Run.....	0.58	Steep, rocky slopes.....	213	3.49	0.039	0.045	5 560	8.62	200-ft. course, poor conditions.
6	Fishing Creek.	18.2	Hilly agricultural lands.....	216	3.85	0.013	0.035	172	0.27	Entire basin not covered by storm; main tributary, Beaver Creek, not flooded.
7	Green Branch.	1.66	Steep, rocky slopes.....	126	3.82	0.055	0.040	1 595	2.48	75-ft. course, fair condition.

The results of Mr. Critchlow's measurements are given in Table 2, from which it will be seen that the conditions surrounding the determination made on Bulls Run were poor, the first inference being that the rate of run-off of 5 560 sec-ft. per sq. mile from 0.58 sq. mile is too high. The speaker has not discarded this measurement, however, as it was made in nearly the center of the area of heavy precipitation, and it seems reasonable to expect a higher

rate of rainfall as well as of run-off there than at other points in that area. He has arbitrarily discounted the discharge by 25 per cent. The resulting figure, 4 160 sec-ft. per sq. mile, corresponds, at 100% run-off, to 6.5 in. of rainfall per hour, which is not unreasonable for the center of this storm, the rates indicated by the kettle being 3.1 in. and by the skiff 3.8 in. per hour. The measurement on Manns Run also gives a run-off rate in excess of the rainfall rate indicated by the catch at the skiff, the difference not being sufficient, however, to warrant rejection.

Frequency.—In the eastern half of the United States practically no gauges are used on small streams, owing to the slight economic importance of their flow. Many gauges have been established within the past fifteen years in the arid regions, where it is no longer unusual to see an automatic register chart showing the complete oscillations caused by a cloudburst flood. However, the records so available are too short to be indicative of the frequency of recurrence. The best that can be done is to study the history of small streams traversing some of the older cities. The record of Erie, Pa., mentions a flood on September 13th, 1878, when Mill Creek swept away dams, culverts, bridges, and caused the loss of two lives.* The rainfall recorded at Erie was 5.11 in. on September 12th and 13th, 1878, but no data are available as to what fell within the area of excessive precipitation. Later well-known floods that caused havoc were those of May 17th, 1893, and August 3d, 1915, the latter finally arousing the citizens to the necessity of providing relief.

The cloudburst flood on Willow Creek, that destroyed Heppner, Ore., on the night of June 14th, 1903, seems to have had a precursor in 1883, before the town came into existence. In 1888, a flood fully as serious as that of 1903, came down Hinton Creek, which empties below Heppner.†

Jones Falls, a small stream which flows through Baltimore, Md., has perhaps the longest available record of cloudburst floods. The first flood recorded is that of July 30th, 1754, the details of which are meager. The flood of October 5th, 1786, occurred in the late evening and was very destructive to property of all kinds. Other similar floods occurred on August 8th, 1817; July 14th, 1837, noted for the many lives that were lost; August 24th, 1842; June 12th, 1858, said to have equalled in destructiveness that of 1786; May 11th, 1860; July 24th, 1868, which swept away stone and brick bridges across Jones Falls; and July 23d, 1887. Of recent years, the magnificent sewer system of Baltimore, in which Jones Falls is made to flow in a covered channel through the city proper, has prevented flood damage. In about 150 years, nine floods occurred at intervals averaging 17 years. This compares closely with the Erie record, in which the floods were separated by 15 and 22-year intervals.

Cherry Creek which flows through Denver, Colo., is a well-known offender. It has a drainage area of about 400 sq. miles, and belongs to that class of streams in which extreme flood stages may be caused by long protracted rains, or by rains of the cloudburst type. The report of the Cherry Creek Flood Commission shows that floods caused primarily by rainfall of the

* Report of Chief Signal Officer, 1879, p. 583.

† U. S. Geological Survey, *Water Supply Paper No. 96*, by E. C. Murphy.

cloudburst type occurred on May 19th, 1864, May 22d, 1878, July 26th, 1885, and July 14th, 1912, averaging 18 years apart.

Another long record is that of the old City of York, Pa., through which flows Codorus Creek, a stream draining an area of 223 sq. miles. Selecting the most disastrous floods on this creek, commencing with 1744, the record shows a total of twelve. Of these, that of March, 1784, was caused by overflow from an ice jam, and that of February 21st, 1822, by heavy rains falling on 18 in. of snow. The remaining ten appear to have been caused by extraordinary rains. Excluding the flood of 1772, concerning which little appears to be known, the occurrences were, as follows: In the summer or fall of 1744 and, again, in 1758, both floods being described as having caused great damage to the crops of the pioneer settlers; October 5th, 1786, when storms of unusual intensity swept various parts of Pennsylvania and Maryland; August 8th, 1817, when $200 000 damage was done in York County, the rainfall being said to have totaled 8 in. in 13 hours; May 29th, 1821, said to have been less destructive than that of 1817; October 8th, 1847, caused by extraordinary torrential rains; July 19th, 1850, which exceeded in height any flood since 1822; and June 25th-26th, 1884, said to have exceeded that of 1817, and to have been caused by a rainfall of about 12 in. in 7 hours. This flood swept away all the bridges in York, then a prosperous city. It was shortly followed by the flood of July 5th, 1884, caused by cloudburst rains within the Codorus Creek basin, producing unprecedented stages in some of the smaller tributaries. These nine floods occurred at intervals averaging 18 years. The speaker submits this record because of its value in studying the frequency of recurrence of extraordinary floods in a stream draining an area of about 200 sq. miles. The extent to which the cloudburst run-off figured in these floods cannot be definitely ascertained. It can only be inferred from the descriptions now available that precipitation of the cloudburst type did accompany these storms.

The dates mentioned are useful in throwing light on the recurrence of cloudburst floods, but without specific information as to peak run-off rates they are of small value to the engineer. They indicate, however, that cloudburst floods are more common than is usually conceded.

Flood Control.—Small streams have not been taken seriously enough in the past, and the property that has been placed within the reach of their flood waters can be protected, as a rule, only at considerable expense. The successful curbing of unruly small streams by some communities is encouraging, and it is hoped will find following elsewhere. The municipalities of Harrisburg, Pa., and Watervliet, N. Y., were among the first to adopt the system of retarding-basin control, and it has been so successful that reports concerning these works are scarcely ever heard, a sure sign that they are working properly. In the Miami Valley, the catchment areas of the streams now under control were, for the most part, too large to be within the class of streams dealt with in this paper. Nevertheless, computations were made to make sure of their safety in case of excessive rates of precipitation in short periods of time.

The Germantown and Lockington Dams each control streams draining areas of a little more than 250 sq. miles. These are in the borderland of

the class of streams herein discussed. Although primarily designed to take care of the run-off from storms like that of March, 1913, and greater, these dams are capable of controlling floods of the cloudburst type.

In some localities, because of local conditions, channel improvement and levees may be the only form of protection that can be provided. The retarding-basin system, however, offers special advantages for handling this class of floods, as the total flood run-off is not large and does not require basins of great size.

Design of Structures.—Protection of property against inundation is often a problem, in which the value of the property and other financial considerations limit the size of flood for which protection shall be made. Waterway to pass maximum floods cannot always be provided under bridges, but bridges can be built to withstand the onslaught of great floods without failing. Culverts over small channels often have inadequate waterway when the conditions would make adequate waterway possible without unreasonable expenditures. Whether it is better to permit highways and railroad embankments to be washed out, at long intervals, rather than provide costly structures over small streams, is more a matter of maintenance than of engineering, the damage to adjacent property being important in such considerations. Good engineering would seem to demand adequate structures, unless the road or railway is of temporary character.

In the design of spillways, it has been considered proper to build for the largest flood, and often an additional factor of safety may be introduced. As more data on cloudburst floods become available, the tendency is to build larger and larger spillways, and there are cases where economy demands that such requirements be set aside in order to keep within reasonable limits. Should the engineer's conscience be flexible, or should he insist on providing a structure that will be safe under the ultimate conditions that Nature may impose? Is it better to inform a client that a safe structure cannot be built at the chosen site except at inordinate cost, and that, therefore, the project should be abandoned? Ingenuity and common sense, the two main attributes of engineering skill, can, in most cases, find a way out of such dilemmas. To permit the client to assume the risk of building an inadequate structure is not good engineering, nor will it save the engineer's reputation in case failure occurs.

Special considerations should apply to the remodeling of existing structures, where frequently radical changes are impracticable of introduction. Again, structures the failure of which can in no way imperil life or property now or at any future time, require no special safety factors.

Psychology.—The impressions made by flood catastrophes, however vivid at the time, soon fade from memory. An occasional flood, like that at Johnstown, Pa., becomes historical through the medium of school books and literature. Other floods equally serious from the engineer's point of view have been forgotten by the public. The Heppner, Ore., and the Oil City disasters are seldom mentioned. Few people residing in San Antonio, Erie, Baltimore, or Denver, have any appreciation of the frequency with which their cities are visited by cloudburst floods, and this is true of hundreds of other towns in

the United States. The hydraulic engineer cannot afford to forget; it is his business to provide himself with records that will enable him to make sound recommendations for flood control. For the larger rivers of the United States he is in a fair position to do so, although the records for many rivers leave much to be desired. For the smaller streams, however, the absence of reliable data is appalling. Many years ago, the Pennsylvania Company adopted the policy of systematically elevating its tracks above the highest known flood marks. Within the State of Pennsylvania, it has succeeded in doing so except for a few branch lines of minor importance. Thus, in large measure, it has freed itself from overflow by the flood-ridden rivers of that State. Not so, however, as regards the small streams—these continue to exact their annual toll of washouts. This is true of other railroad systems. In 1883, a cloudburst took out a large embankment on the Delaware, Lackawanna and Western Railroad about 1 mile east of Moscow, Pa. Again, on June 10th, 1892, a fill nearly 50 ft. high was washed out on this railroad near Moscow, the culvert at that point being unable to carry the flow from a cloudburst discharged by a drainage area of less than 5 sq. miles. On July 10th, 1914, the same fill went out by the same process. The Railroad Company restored the fill to its former lines, but replaced the culvert with a 16-ft. semicircular arch. The question may well be asked: Should bridges of ample span have replaced the culverts, or was the rebuilding of the embankments the logical thing to do?

In closing, the speaker wishes to state that he has purposely refrained from introducing into this paper a large collection of compiled data. Rather, it has been his endeavor to present as much as possible the results of his personal observations, covering a period of twenty years and extending over a large part of the United States. It is his belief that the present chaotic state of knowledge of cloudburst floods is due simply to lack of data, and that much could be accomplished toward placing this subject in its proper light if some agency would take on itself the systematic collecting of information pertaining to cloudbursts. The relation between topography and cloudburst frequency is a phase of the problem that merits especially exhaustive study. It is not enough to know how great rates of run-off can occur. It should be possible to foretell whether any given locality is subject to such rates by reason of its surrounding topography. It would be just as poor engineering to provide a structure of maximum dimensions in a locality that is immune from cloudbursts, as to build an inadequate structure in a locality where they are bound to occur often.

STANDING WAVES IN RIVERS

By Nathan C. Grover,* M. Am. Soc. C. E.

Standing waves in rivers are due to several distinct causes and are known by many different names. They differ greatly in appearance, reach maximum heights of 20 ft. or more, and may be fixed in position or may move either with or against the current. In some forms, they are common and attract no attention; in others, they are unusual, spectacular, and, under some conditions, destructive to property and even dangerous to life. The four conspicuous and generally recognized types of standing waves are the hydraulic jump, caused by a sudden checking of the velocity of swiftly moving water; the sand wave, caused by moving sand on the bed of a stream that is heavily burdened with silt; the tidal bore, caused by the flow of the tide into a river; and the standing flood wave, caused by a sudden influx of a large quantity of water into a river channel.

Hydraulic Jump.—The hydraulic jump which is due to a sudden checking of the velocity of rapidly moving water and a corresponding increase in cross-section, may result from a sharp decrease in slope or from an abrupt increase in resistance to flow caused by an obstruction or a series of obstructions in the channel. By "facing" the current producing the wave and having no linear motion, it differs radically in appearance from other forms of standing waves herein described. It will occur where swiftly moving water flowing over a dam strikes the more slowly moving water in the pool below, where the jet of water from an orifice that is partly or wholly submerged strikes the relatively still water into which it flows, and where there is an obstruction or an abrupt increase in frictional resistance to flow. Practically all roughness of the surface of flowing water, except that caused by wind or by a sharp drop in the channel, is a manifestation in some degree of the hydraulic jump, although not commonly so called.

This phenomenon below a spillway dam and other hydraulic structures has been described and analyzed mathematically by Karl R. Kennison,† M. Am. Soc. C. E., Julian Hinds,‡ and S. M. Woodward,§ M. Am. Soc. C. E. In these descriptions, two important features of the hydraulic jump have been emphasized—the danger of the destructive effects of the jump on hydraulic structures and the possibility of utilizing the jump for removing part of the kinetic energy of the moving water. The destructive effects have long been recognized, and allowance has been made for them in the design of canals and flumes and of the foundations of overflow dams. The possibility of using the jump for dissipating energy is apparently much less widely understood and may be of sufficient importance to warrant further experimental study.

* Chf. Hydr. Engr., U. S. Geological Survey, Washington, D. C.

† "The Hydraulic Jump in Open Channels at High Velocities", *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 338–420.

‡ "The Hydraulic Jump and Critical Depths in the Design of Hydraulic Structures", *Engineering News-Record*, Vol. 85, 1920, pp. 1034–1040.

§ "Theory of the Hydraulic Jump and Backwater Curves", Miami Conservancy District Technical Reports, Pt. 3, 1917.

Sand Waves.—Sand waves occur at high stages of streams heavily loaded with silt and are caused by the movement of considerable waves of sediment on the stream bed. The observed wave is, however, a wave of water, its causal wave of sand being entirely submerged. The sand wave may be due to the same causes that produce the hydraulic jump, but its occurrence is so striking and its appearance is so different that it is treated as an independent phenomenon. It appears to be due to a sudden checking of the velocity of the swiftly moving water, but it is caused by an obstruction that is in motion and that vanishes and re-appears with the dissipation and formation of waves or dunes of sand on the bed of the river.

These phenomena have been described by Gilbert,* who has called the waves of sediment on the bed "rhythmic dunes" and has made a distinction between "dunes", which travel down stream, and "antidunes", which travel against the current. The motion of the antidunes is caused by the deposition of sediment on their up-stream slopes and simultaneous erosion from their down-stream slopes. The dunes cause only slight undulations of the water surface, but the antidunes which travel much faster than the dunes, may cause waves of considerable magnitude.

R. C. Pierce,† Assoc. M. Am. Soc. C. E., who observed many sand waves in the San Juan River in Southern Utah, has described them as resembling in appearance "the waves thrown up by a stern-wheel river steamboat." He observed sand waves at least 6 ft. high, but they were generally only about 3 ft. high, measured from trough to crest. The length of the wave from crest to crest in the deeper sections of the river was from 15 to 20 ft. He describes their appearance as follows:

"At one moment the stream is running smoothly for a distance of perhaps several hundred yards. Then suddenly a number of waves, usually from six to ten, appear. They reach their full size in a few seconds, flow for perhaps 2 or 3 min., then suddenly disappear. Often, for perhaps half a minute before disappearing, the crests of the waves go through a combing movement, accompanied by a roaring sound. On first appearance it seems that the wave forms occupy fixed positions, but by watching them closely it is seen that they move slowly up stream. In the narrow parts of the stream the waves may reach nearly the width of the river, but in the wider parts they occupy smaller proportional widths. Usually they are at right angles to the axis of the stream, but at some places, particularly in the wider parts of the river, they may suddenly assume a diagonal position, moving rather rapidly across the stream in the direction toward which the up-stream side of the wave has turned."

Tidal Bore.—The tidal bore seems to be peculiar to a region of large tidal fluctuation and is formed in a tidal estuary where there are marked restrictions of channel or in a large river of strong current that enters the ocean. The effect is a piling up of the advancing tide on the lower layers, which are retarded by the normal current of the river. Many of these bores are destructive and are serious obstacles to navigation and shipping. The name "Amazon" is said by some authorities to be derived from an Indian word

* "The Transportation of Débris by Running Water", U. S. Geological Survey, *Professional Paper 86*, 1914, pp. 11, 242–243.

† "The Measurement of Silt-Laden Streams", U. S. Geological Survey, *Water Supply Paper 400*, 1917, pp. 41–43.

meaning "boat destroyer", because of the dangerous tidal bore about 100 miles above the mouth of the river. This bore occurs in a stretch of the river where the channel is obstructed by islands and the depth is not more than 4 fathoms. It advances at a rate of 10 to 15 miles per hour with a breaking wall of water from 5 to 12 ft. high. On applying Merriman's formula,* $v = \sqrt{g\,d}$, to these rates it is found that a velocity of 10 miles per hour (15 ft. per sec.) corresponds to a depth of wave of 7 ft. and that 15 miles per hour (22.5 ft. per sec.) corresponds to a depth of wave of 16 ft.

The tidal bore in the Ganges is noted for its magnitude and destructiveness. On the Hugli Branch of that river, the bore advances 70 miles in 4 hours, or at the rate of 25 ft. per sec., indicating a height of wave of about 20 ft.

Standing Flood Wave.—The standing flood wave occurs in a shallow stream when a large quantity of water reaches the channel while the stage is low. It may be caused by the sudden release of water through the failure of a storage dam or by heavy local rainfall. The same quantity of water reaching the stream at a higher stage would cause an increase of stage and discharge, but might not make a standing wave.

The standing flood wave is caused by the lag of water on and near the bed of a stream, due to the greater resistance to flow on the bed than on the water itself, which is shown by the ordinary curve of velocities in a vertical section of a river. In this respect, it is closely analogous to breakers on a beach and more remotely to the tidal bore. As a result of the lag of water near the bed, the swiftly moving water in the upper part of the cross-section of flow is continually overtaking the more slowly moving water below. A standing flood wave will be caused only when the quantity of water originally in the stream is small in comparison with the quantity in the sudden flood. Therefore, it is, perhaps, seldom the direct result of rainfall in humid regions, but it is a characteristic feature of streams in arid and semi-arid regions where low-water flow is small and precipitation has the intensity of the so-called cloudburst.

The flood wave appears as a nearly vertical "wall of water" with much splashing and foaming in front, caused by the continual falling of the faster moving water near the top over the slower moving water near the bottom. The water which thus falls becomes a part of the bottom layer of water, and other water from above overtakes and falls over it. The quantity in the bottom layers, of course, must be constantly augmented from above as the flood wave progresses, because the top layers must supply not only the water for their own extension down stream, but a large part of the water needed for an equally rapid extension of the slower moving bottom layers.

If a sudden rush of water should occur in a frictionless channel, the lower layers, acting under hydraulic pressure, would move forward more rapidly than the upper layers. In such a theoretical channel, nothing resembling a standing wave could form, as the piling due to the continual overtaking of the lower layers by the upper ones would be replaced by a flattening due to the more rapid movement of the water near the bed. It appears, therefore,

* "Treatise on Hydraulics", 8th ed. (1908), p. 342.

that friction of water on the bed is essential to the occurrence of a standing flood wave.

Merriman* has shown the mathematical conditions controlling the production of the standing flood wave. He finds that it will occur when the velocity becomes equal to $\sqrt{g\,d}$, provided c in the Chezy formula, $v = c\sqrt{r\,s}$, is less than $\sqrt{\frac{g}{i}}$, in which d is the depth of water, i is the slope of the channel, and g is the acceleration of gravity. The condition of this proviso will prevail only if c decreases—that is, if the frictional resistance to flow increases—as the slope of the bed increases. In other words, the steeper the slope, the greater the frictional resistance essential to form a standing flood wave.

Although, in the aggregate, many more standing flood waves are formed by excessive precipitation than by the failure of reservoirs, rainfall does not generally produce the best conditions for such waves, because water derived from rainfall normally reaches the large drainage channels in part through many tributary channels that are more or less separated and in part as sheet flow over the surface of the ground, the result usually being a gradual increase of water in the stream bed rather than a sudden rush of water from one source. The natural increase down stream in channel capacity and the effective storage of water required to fill the channel and cover the flood-plains also tend to counteract the conditions that promote the formation of a standing wave. As a result, standing flood waves are formed from rain only when the local precipitation is unusually intense.

Unlike the tidal bore, which recurs periodically in the same stream, the standing flood wave cannot be anticipated and may recur in the same stream only at widely separated times. When it occurs, there are generally no facilities available for observing its velocity, its height, or its variation in velocity, height, or form, as it moves down a channel of varying width and slope. Consequently, few, if any, exact observations of the depth and velocity of standing flood waves have been made, and although such waves resulting from rainfall are frequently mentioned in general terms in both engineering and popular literature, there appear to be few published descriptions of their height and velocity.

Optimum conditions for standing flood waves occur in connection with the failure of reservoirs, of which the Lower Otay Reservoir, in California, afforded a typical example. The failure of that reservoir produced a standing flood wave that has been described as a huge wall of water from 6 to 20 ft. high, traversing the 10 miles from the dam site to Palm City in 48 min., corresponding to an average velocity of about 18 ft. per sec. This velocity corresponds to an average value of d (depth of wave) of about 10 ft. in the formula, $v = \sqrt{g\,d}$. Similarly, if the average velocity of wave of the Johnstown flood was 25 ft. per sec., as has been stated, the value of d found from this formula is 19.4 ft.

The four types of standing waves described pertain only to rivers and should not be confused with breakers, with the infrequent "tidal waves" of

* "Treatise on Hydraulics", 8th ed. (1908), p. 344.

the ocean caused by volcanic action or earthquakes, with unusual fluctuations of tides in bays or in estuaries into which no large river empties, or with those most common waves of all which are caused either directly or remotely by wind. These waves have been fully analyzed and described in engineering literature by several writers, and mathematical statements of certain of their characteristics have been presented to the Society by Karl R. Kennison, Assoc. M. Am. Soc. C. E., in the paper, to which reference has been made, and by Mr. R. D. Johnson.*

Obviously, however, the hydraulic jump and tidal bore form the basis of most discussions of standing waves in rivers, as they may be studied systematically at the will of the observer. In some respects, the sand wave is interesting, but it is of little importance to engineers, because it occurs only in a few rivers that are heavily loaded with silt and that are, therefore, generally not utilized in hydraulic developments. The standing flood wave, as it occurs in natural channels, although seen many times and described in general terms, has not been and from its nature cannot be studied in detail, either as to causes, form, progress, or effects. These are good reasons why engineering literature contains no reliable information in regard to this spectacular and destructive phenomenon.

* "Surges in an Open Channel", *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 112.

FLOOD PROBLEMS IN CHINA

By John R. Freeman,* President, Am. Soc. C. E.

The inclusion of notes on the flood problems of China in this Symposium might at first view not seem helpful toward the solution of certain flood problems of America, for which engineers are striving; nevertheless, there are certain important facts about training rivers to cut their beds deeper where they flow over deep alluvial deposits, recently discovered in the course of the speaker's researches in China, that may prove of practical application in America and in other parts of the world.

Floods in China imperil the lives of a greater number of people than anywhere else on earth. Many of the great famines of China are caused by floods which, spreading to shallow depths over vast areas, even thousands of square miles, drown the domestic animals, destroy the growing crops, and make impassable the few poor roads, so that relief cannot be brought in. Many other of the great famines are caused by an occasional shortage of rain in districts where irrigation from controlled rivers or from storage reservoirs would insure food to lessen this starvation menace. In brief, there is no part of the world where the science and arts of hydraulic engineering could do more for humanity than in China.

The regulation and guiding of certain of China's great rivers in channels better adapted for flood control and for navigation present some of the most interesting problems of hydraulic engineering that can be found anywhere in the world, and the speaker has been led, through personal observation, to believe that the control of the rivers which cause these great floods is practicable to a large extent by modern engineering, and that it can be carried forward whenever China's domestic political affairs become straightened out and the inertia of extreme conservatism is overcome.

Until twenty or thirty years ago, the world outside of China knew little about these terrible floods, because means of communication within China were so poor that a knowledge of the facts filtered out slowly and imperfectly. For example, few now know that as recently as thirty-five years ago more than 1 000 000 people (some say 7 000 000) perished by drowning and starvation, resulting from one flood which came from a break through the south dike of the Yellow River, about 20 miles above Kai-feng City, and flowed southeasterly over a belt of nearly level country, perhaps 20 or 30 miles in width by 150 miles in length, into the Huai River. No accurate description of this outbreak or of its wide devastation appears to have reached Shanghai until after many weeks. Few to-day realize the awful devastation that followed the floods which, in August and September, 1917, surrounded Tientsin, the great commercial port of North China, submerged a large part of 15 000 sq. miles of delta plain, containing 105 cities and 17 000 of the little Chinese farm-villages, numbered 5 600 000 sufferers (by partial count of the magistrates), ruined crops estimated by the head of the Relief Commission to

* Consulting Hydraulic Engr., Providence, R. I.

have been worth $22 000 000,* and caused a property loss several times this sum. Few who have not traveled in the Chinese deltas have any conception of the density of population or of the vast number of these little villages dotted everywhere over the fertile delta plain.

Floods that in any other populous country would receive world-wide attention occur two or three times in almost every decade at one part or another of the great eastern Chinese delta plain. In July, 1919, while the surveyors on the Grand Canal were in the field, there was a flood from an outbreak through the Yellow River dike, that ruined the crops over about 125 sq. miles

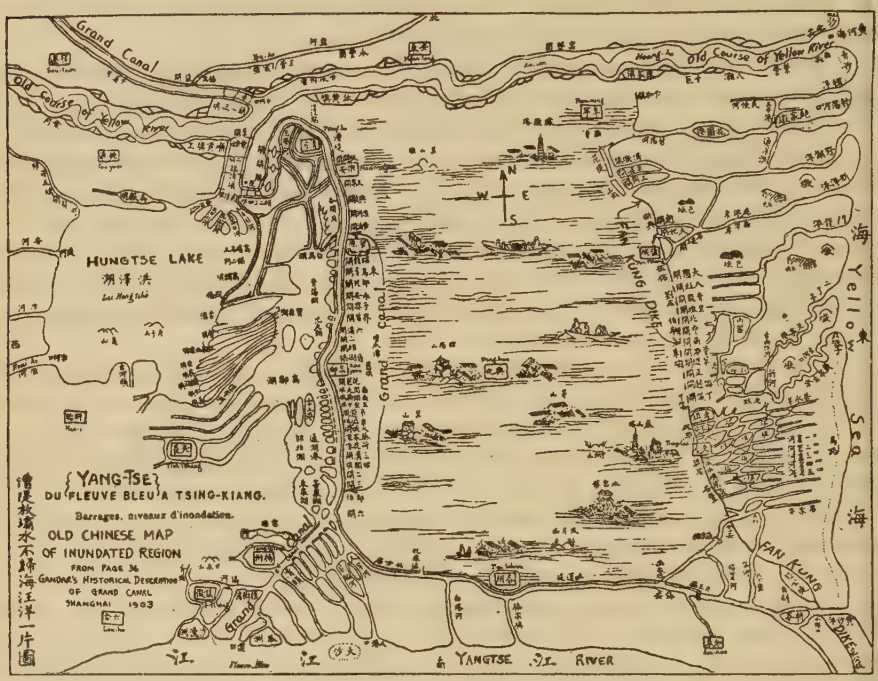


FIG. 3.—CHINESE PICTURE MAP OF KIANG-SU FLOOD.

of fertile land, supporting 560 villages and 217 000 people, and caused damages (as estimated by the local magistrates) to the extent of $350 000; and in July, 1920, a hundred miles away from the area just mentioned, following sudden rain, there was an outbreak through the dike of a little river, that spoiled crops said to have a value of $250 000, which outbreak was observed by the principal American field engineer. These two floods, of 1919 and 1920, were so commonplace in comparison with the great floods which always threaten, that as far as the speaker could learn, they escaped all notice in the several important English language newspapers published in China. The summer of 1921 brought a repetition, for perhaps the hundredth time in his-

* The distinction between U. S. gold and "Mex." silver dollars, an ordinary basis for estimating values in China, which are worth about one-half the gold dollar, has to be made plain in publications dealing with China. All values in this paper are calculated in U. S. gold dollars, at two silver for one gold.

tory, of great floods in Kiang-su Province and of another break in the Yellow River dike somewhere on the lower river, which is said to have caused great damage and a shifting of several miles in length of its channel, although no description of it has yet reached America.

In Kiang-su Province, along the branching outlets of the Huai River, north of the Yang-tze River and east of the Grand Canal, there is a flat delta plain of extremely fertile land extending nearly 100 miles north and south by about 50 miles east and west, which vast area, time after time, has been covered with water from the Huai floods, deep enough to drown most of the domestic animals, ruin the crops, and consign multitudes of people to starvation. An old Chinese picture map of this region in flood is shown in Fig. 3.* The area shown flooded covers about 4 500 sq. miles. This district which now supports an impoverished population said to number about 2 000 000, could be made, by means of construction works which the speaker believes practicable, to support, in comfort and safety, a population far greater than at present.†

The great industrial leader, Chang Chien, says, in his report of 1919, urging National action, "Within this district along the Huai River below the City of Wu-hu, in the past thirteen years, there have been six serious famines affecting thousands and thousands of people."

In the speaker's two brief tours of China, he has seen enough of some of these regions menaced by flood to convince him that no country has greater need of careful engineering research and sound cautious advice; and from his studies for the improvement of the Grand Canal and of the Yellow River, he is confident that great relief can be secured by an expenditure which relatively is not large and which can be forthwith recovered in the increased value of the land reclaimed. The construction can be performed mostly with hand implements, by native labor, which badly needs employment, at a smaller cost than by steam shovels, industrial railways, or dredges.

No part of the world gives greater opportunity for beneficent service to suffering millions of worthy, kindly, industrious people, by means of engineering skill, combined with a substantial construction loan to the Chinese Government.

Concentration of the Floods

The outline map of China, Fig. 4, shows that run-offs from the principal river drainage basins are gathered into remarkably few main outlets to the sea, there being, within 2 500 miles of main coast line, measured without including the sinuosities, only five or six outlets to carry off the discharge from a catchment area about equal to the United States east of the Rocky Mountains. Naturally, the flood characteristics of these widely separated districts differ greatly.

* From Fr. Gandar's "Histoire du Canal Imperial."

† Ex-Minister Reinsch says in "An American Diplomat in China" (p. 60): "The people of the Huai Region, secure and affluent, might be easily increased by twenty million living heirs of a fifty-century old civilization.

Almost nothing was known of the detailed topography of the interior of China, until within the past 20 years, except from the journals written along single lines of travel by Abbé Huc, Pumpelly, and Richthofen; and precise information about the floods of these five great river systems is still limited to a few areas, large in themselves, but small compared with the whole of China.

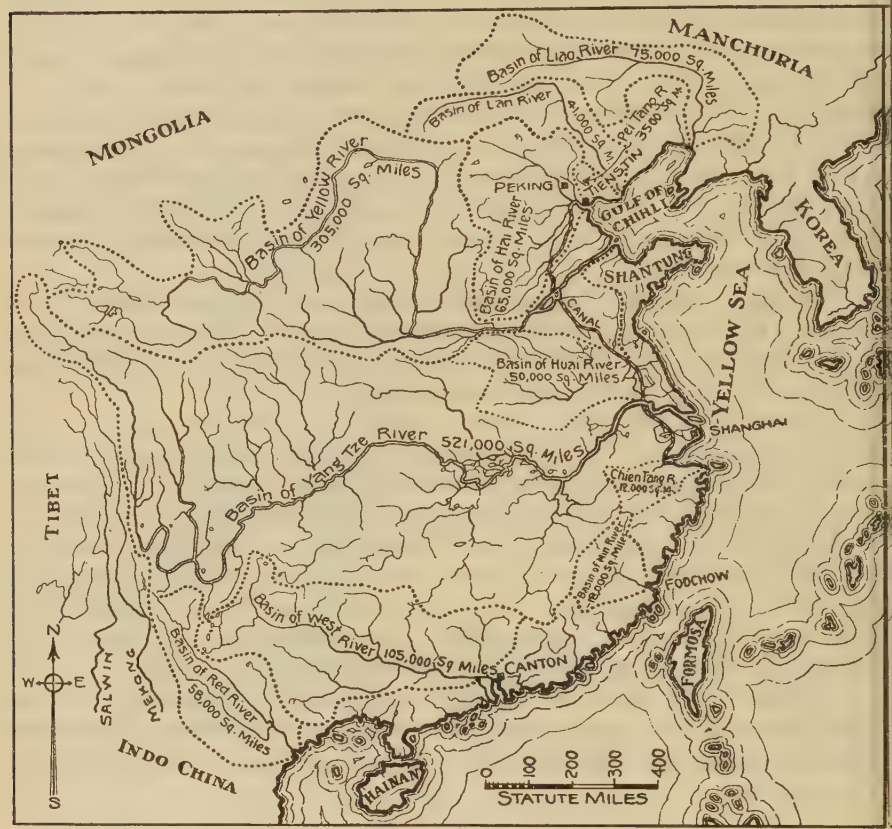


FIG. 4.—DRAINAGE MAP OF CHINA.

These particular flood areas about which something is known are mostly near the commercial centers. Doubtless, these districts are the worst afflicted, because they are within the vast delta plains where a flood can spread. Back from these deltas near the sea, Chinese topography presents vast areas of hill and valley, which doubtless possess all ordinary varieties of the flood problems found in other parts of the world, that come from torrential rainfall and run-off in relatively narrow valleys, with great and rapid range from normal to flood height; for China has many rainfall records of 8, 12, and even 15-in. depths falling in a single storm, which equal and exceed that in the great storm of the Miami Valley.

This paper does not deal with those floods of upland regions. They concern fewer people and probably their control is to be found mainly in the slow processes of systematic forestry, or perhaps by barrages, such as are used in certain Swiss valleys, for preventing the landslides caused by torrents, from under-cutting the steep hillsides.

Re-Forestation and River Training

The situation in general for flood relief requires much more than systematic forestry, although the sooner widespread re-foresting of the hills is vigorously undertaken, the better. The thickly settled lowlands of China are almost treeless, except small spots around the graves of ancestors, and during the past few centuries the uplands far back along the rivers have been robbed of their natural protection; it is certain that the cutting of the forests has had much to do with the present disastrous conditions. For example, up stream from Tientsin, vast areas of hills, from which floods descend, are said to have been stripped of their natural forest cover centuries ago and, consequently, their humus and their porous top soil have been largely washed away and the subsoil cut into gullies by torrential rains. Many of these steep slopes which, although unfit for growing grain, might be made to grow much needed timber, are now so bare that they shed water almost like a tiled roof.

Although forest litter, humus, and forest shade will unquestionably delay run-off from rain and snow in China as elsewhere, the disastrous delta floods on the Yellow and the Huai Rivers come largely from climatic and topographic conditions far beyond forest control. Forestry would protect the hill slopes and aid in moderating the lowland floods, but in the vast Chinese deltas, quicker positive safeguards are needed by means of channel control, new channels, and strong dikes, and these methods of relatively prompt relief seem feasible to the speaker whenever funds and proper direction can be had.

It seems probable that the new works of river training and dike building needed for the complete protection of certain large areas, which works, although extensive, are of a simple character, will cost annually far less than the present unreliable structures, and it also appears probable that the value of waste land that can be reclaimed by these new works could promptly be made to pay back to the Government, their entire cost and, in addition, bring prosperity to a vast impoverished region, and to other regions bring safety from impending disaster. All this complete flood protection, however, is no short and simple task, nor can the works be properly designed until scientific tests and painstaking observations have been made.

The speaker will present, in this paper, various tentative plans for flood relief in certain localities. Although these plans are the result of much study, and observation, he desires it clearly understood that, in their present form, they are suggestive rather than conclusive, and are subject to revision after more complete surveys. They are presented now in the hope of stimulating further investigation in the immediate future.

Some Localities Having Important Flood Problems

From north to south there are five principal localities within these main Chinese drainage basins, shown on the map, Fig. 4, in which flood relief needs to be intensively studied and great works constructed whenever the Government becomes stabilized and capital gains confidence so that means can be provided. These localities are, as follows:

(1).—In the flat, level plains of the drainage basin that has its outlet past the great commercial city of Tientsin. This region suffered terribly in August and September, 1917. An area of about 12 000 sq. miles was submerged, and more than 1 000 000 people, mostly farmers, are said to have been driven from their homes. A property loss of more than $25 000 000 is said to have been incurred in this one flood with consequential damage of perhaps $50 000 000.

(2).—In the delta of the Yellow River, or Hoang Ho, along the river's 400-mile course through the delta. During the historic period of 4 200 years, this region has been ravaged time and again by floods in one place and another. These overflows come from breaks in the great dikes and occasionally cause a wide change in the river's source. So terrible have been these visitations that this river is often called "China's Sorrow". The conditions are aggravated by the silt burden, brought from the vast and easily eroded loess deposits of the Provinces just up stream from the delta, which make the Yellow River probably the muddiest great river in the world.

(3).—The Huai River District in Kiang-su Province below Wu-hu. About 5 000 sq. miles of this peculiarly low and level delta region is said to be frequently inundated, with famine often following flood. The land is exceptionally fertile and normally gives two crops per year, so that, for thousands of years, intensive cultivation has been continued in spite of disasters which, sometimes, have brought death to thousands and, at other times, have merely destroyed the second of the two yearly crops.

(4).—Along the Yang-tze River below Ichang. Here, in the Grand Gorges, the river floods are said to rise 80 ft., and below Hankow the flood discharge is said to reach about 3 000 000 sec-ft., which is about 50% larger than the greatest flood discharge of the Mississippi River at New Orleans, La. The Yang-tze River, although turbid, brings down less silt than the Yellow River. Its delta, therefore, grows more slowly. Its whole regimen has become well established, and the flood problems are mainly those of ordinary dikes combined with river training on a mighty and forbidding scale, because of the depth and volume of water. The charts show dozens of spots where the swirl of the flood cuts pools more than 100 ft. deep, and it is no light problem to work out shore protection that will hold so deep a bank of soft fine-grained river silt when attacked by the under-cutting of such a flood.*

(5).—In Southwestern China the valleys of the North, East, and West Rivers, in the country around and back from Canton. Near the important City of Wu-chow, the great West River is said to rise 50 and even 80 ft. in extreme floods, and at the rate of 1 ft. per hour.

* The Mississippi River, at Greenville, Miss., rises about 50 ft. in extreme floods, and has a flood depth in main channel upward of 100 ft.

(6).—The improvement of navigation also presents incidental problems of flood scour, in addition to those of protection from inundation. There are particularly great and difficult problems of river training for improving navigation, and problems of utilization of flood flow for scour of channels for harbor improvements, to be found on the Yang-tze near Shanghai, on the Min River near Foo-chow, and all along each of these five or six great silt-bearing rivers of China proper.

The great depth of the silt deposit in these Chinese deltas gives unlimited scope for the river to scour its bed deeper when confined in a generally straight and narrow way.

(7).—Outside the limits of China proper, to the northeast in Manchuria, there are great flood problems along the Liao River which rises in the Mongolian slopes and brings down silt that forms a harbor bar at New Chwang. On the other side, 17 000 miles away, in a straight line to the southeast, in French Indo-China, the lower course of the Red River presents serious flood problems which the French Government is said to be considering.

Besides the large problems cited, there are many smaller ones in drainage basins from 20 by 100 miles to 40 by 100 miles in extent, that are almost too small to show on the map of all China, to which floods may come once in 10, 20, or 40 years, that are terrible enough to the many people living on little farms or in little villages in their path. Examples of them were found in course of the Grand Canal surveys along the Wen River and in the Red Cross reconnaissance on the Yi River. Doubtless, there are a hundred of these minor areas in fertile and densely populated parts of China, each of which has its own important flood problem which can be solved so as to bring a greater measure of security and happiness, whenever the Government becomes stabilized, a broader community spirit developed, and when the multitude of young Chinese now studying modern engineering methods at home and abroad, are given the means wherewith to work.

Ancient Chinese Flood Protection

The Chinese have had to contend with the problems of flood control by dikes during their whole historic period of about 4 000 years and have developed some wonderfully good technique in many matters of dike building and in the repair of dike breaks. Although in the science of river training, they (and all engineers) still have much to learn, they have, in many great works, shown skill as hydraulic engineers. They have had some of the most difficult problems in the world with which to contend.

One of their most venerated men, of the half-legendary days of 3 000 years ago, was Yu, their great hydraulic engineer, to whose memory many temples were built and who, after a period of trouble, was entrusted with the conservancy of rivers throughout the country. He regulated the waters so wisely, it is said, that with his precepts faithfully followed there was no serious trouble for more than 1 000 years until "the period of the warring States". Tradition says, "Yu labored thirteen years, sparing neither trouble nor fatigue, nor even once entering his own home, though he passed three times before its door." That "he had boats for travel by water, chariots for

travel by land, sledges for mud, with relays of men to draw them." "He dug nine great channels to conduct the waters to the sea," and he is supposed to have organized the building of great systems of dikes. In the great fertile Province of Szechwan, many travelers have described great irrigation works and flood channels planned more than 2 000 years ago by an engineer, whose rules are said to be still implicitly followed. Tributary to the Grand Canal, at Tai-tsun-pa, there is a feeder dam, which the speaker has examined, of masonry resting on small piles driven into a soft sand foundation, that was admirably built about 500 years ago. The sea wall, of coursed cut stone, 25 ft. high, that protects the coast for many miles easterly from Hangchow, is a structure built about 400 years or more ago, of which any modern engineer might be proud. The fact that, during 538 years, the great restless Yellow River was held to its course on a silted bed several feet above the level of the ground on either side, within dikes built of soft friable river silt, carefully consolidated by tamping and protected from the erosion of impinging currents in many places only by groynes and revetments of earth bound together with perishable millet stalks, speaks volumes for the skill and resources of these Chinese "Old Masters of River Control".

Preliminary Surveys in Progress

In all the localities previously mentioned, investigation is now being carried on under the Chinese Government; but everywhere* this work seems to be mostly in the surveying rather than in the engineering stage. It is far easier to make a tolerable survey than to make a good design and then establish confidence in it; and one gets an impression that the authorities having certain of these matters in charge are staggered by the size, complexity, and uncertainty of their problems, or are floundering in a "Slough of Despond", and that some of them may be forced, by pressure of the public demand for "making the dirt fly", into constructions that will not stand the test of time, and of which they themselves may have doubts. The data collected in these surveys and investigations are few in comparison with the vast territories in China affected by flood.

For investigations within the Hai River Drainage Basin, following the great Tientsin flood of September, 1917, the Chinese Government organized the Chihli River Improvement Commission which, under the guidance of English, American, and Chinese engineers, including three members of the Society, has, for about four years, been making topographic and hydrographic surveys, from which data, works may be planned later.

All along the Grand Canal in Shantung Province, the China Grand Canal Improvement Board, under the supervision of American engineers, members of the Society, has made extensive topographic and hydrographic surveys and a reconnaissance survey of the Yellow River's course and its dikes for 200 miles up stream from the Grand Canal crossing. Plans have been devised for the immediate reconstruction of 253 miles of the Grand Canal, whenever this work can be financed; but from lack of funds, owing to the difficulty of selling Chinese bonds under present disturbed political conditions, the enterprise is now marking time.

* Except on the Min Estuary.

Along two or three portions of the 400 miles of the Lower Yellow River within the delta plain, the Provincial Conservancy Boards are making outline surveys of the river's course and of its dikes, but with little attention to accurate hydrography or levels as far as the speaker has been able to learn. Meanwhile, the Chihli River Commission is gauging the Yellow River flow and measuring, at frequent intervals, the percentage of silt that it carries, at the Tientsin-Pukow Railroad Bridge. The daily height of the Yellow River is recorded also at the Peking-Hankow Bridge, 302 miles up stream.

In Kiang-su Province, under the leadership of one of the most remarkable men in China, His Excellency, Chang Chien—classical scholar, Confucian philosopher, captain of industry, and philanthropist, full of the spirit of service to his fellow men—who seeks means for protecting his Province from floods, much of the delta land is being covered by a topographic survey made wholly by Chinese surveyors, that has been in progress for ten years, with ½-m. contours over large critical areas. This is accompanied by occasional, more or less accurate, river-flow gaugings.

For several years, the Harbor Board of Shanghai, controlled chiefly by the English, American, and French commercial interests, with a view chiefly to the improvement of navigation, has been making an excellent systematic and thorough survey of the neighboring Yang-tze River and its small tributary river, the Whang-poo, on which Shanghai is situated, under A. V. H. von Heidenstam, M. Am. Soc. C. E., assisted by E. C. Stocker, Assoc. M. Am. Soc. C. E.

At Foo-Chow, some good river-training work is said to be in progress, mainly for improving the entrance from the sea to the harbor, in charge of J. R. West, M. Am. Soc. C. E.

In Southwestern China, along the great West River and other rivers near Canton, some excellent preliminary hydrographic studies relative both to navigation and flood problems have been in progress for five years, in charge of Capt. Olivecroner, a Swedish engineer.

As a whole, the outlook for important construction, everywhere, is poor, because of internal political conditions, the impossibility of the present Government securing adequate funds, either by internal taxes or outside loans, and by the lack of community spirit, but the speaker believes that sooner or later China will find a way to work out its own salvation, and that the present outlook on stagnation will suddenly change.

Although China proper presents a range of latitude and longitude about equal to that of the United States east of the Rocky Mountains, this paper will be confined to the first three problems previously mentioned, which are found in the northeastern delta plain of China, north of the Yang-tze River, within an area which, on a map of the United States, would about cover a triangle with corners at Buffalo, N. Y., Boston, Mass., and Washington, D. C. First the flood problems of the Yellow River, "China's Sorrow", will be briefly discussed.

Flood Problems of the Yellow River Delta

By far the largest of the Chinese deltas is that of the Yellow River which, with a radius of about 400 miles and an apex angle of about 90°, from its

mountain exit, slopes seaward with wonderful uniformity at the rate of 10 in. per mile measured on a straight line, or about 8 in. per mile as measured along the river's winding course. Through millions of years this delta cone has been built up by deposits of fine-grained silt, brought down by the floods

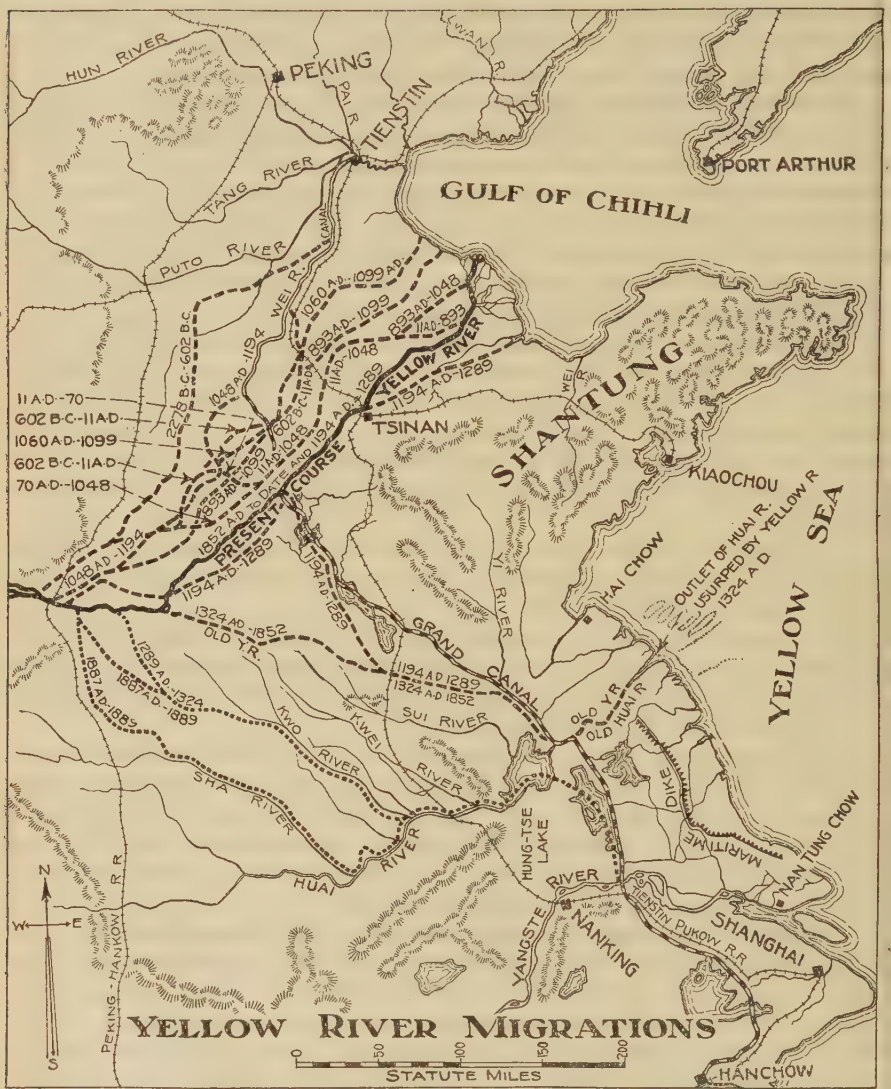


Fig. 5.

from the vast loess deposits of Shansi and other Provinces. The flatness of the surface of this vast delta-cone is the cause of the great width and vast range to which a flood may spread when it escapes from the river's dikes. This flat slope may be due to the extreme fineness of the particles of silt,

which are mostly derived from the erosion of vast beds of loess, supposed to be accumulations of wind-blown dust, that originally came largely from the vast Gobi Desert. A moderate current can carry this fine-grained material a long way.

The Yellow River is not large in volume, compared with other great rivers of the world, although it is about 2 350 miles long, without counting the minor bends, and drains about 305 000 sq. miles. Within the historic period of about 4 200 years, the Yellow River has meandered and shifted from north to south, and back again, through this delta plain, occupying the nine widely divergent channels shown in Fig. 5.

The courses followed in these migrations appear to have first been brought to the attention of the outside world by the American explorer and geologist, Raphael Pumpelly, about 1865, in a series of small maps. The speaker had the map which is presented on a reduced scale in Fig. 5, prepared in China, in 1919, by tracing on a sheet about 4 ft. square, the several courses laid down on a series of Chinese maps brought to his attention by the engineers of the Chihli River Improvement Commission. These maps were the work of a Chinese historian of about 100 years ago, who based them on the investigation of a Chinese author of about 200 years ago.* The Chinese have had some remarkably painstaking scholars and historians.

When a complete topographic map of China on a large scale is prepared, many additional details will probably be secured, because when one travels across the delta, many depressions, sand dunes, abandoned dikes, and other indications of ancient channels are found. Many town and provincial records have been kept for centuries, which note changes and floods with evident care. It is important that all these sources of data be searched and that a complete contour map be made of all the delta, showing all ridges and drainage channels, as a basis for many possible improvements; but the solving of the main problem of flood protection need not wait for this.

Since time immemorial, the river floods have been confined between dikes built of river silt, well tamped into place, which dikes are guarded from erosion, wherever the river threatens, by spur-dikes or groynes commonly built with a facing of loose stone rip-rap over a core of earth and millet stalks, but sometimes built only of bundles of millet stalks, tied together with straw ropes, packed with earth, and pinned down with small stakes. This turning away of an impinging flood by spur-dikes is made easier by the great width of open ground between the dikes, which area is largely made unavailable for agriculture by the danger of flooding. On some of the higher silt banks of the flood-plain between the dikes, the farmer often sows his seed and takes a chance of loss of harvest if the annual flood arrives early.

Excessive Width Between Dikes

In general, along the upper 200 miles within the delta, and as shown in Fig. 6, these inner dikes are from 4 to 8 miles apart, although the necessary channel width at ordinary stages is only about ¼ mile, and for floods, less than ½ mile. The water sometimes spreads in a thin sheet over the elevated

* "Notes on the Tribute of Yu", by Hu Wei, published in 1708.

terrace of silt deposited between the normal shore line and the dike, to a much greater width than ½ mile, sometimes to the entire width between the inner dikes; but there are localities, as at Wei-chia-shan and Chang-kou, where the river happens to be confined between rocky hills projecting through the deep silt deposits, and here it flows in apparent comfort in a channel having a flood width of only about ⅓ mile, without abnormal fall, over a bed easily erodible to any necessary depth. Careful study of soundings and velocity measurements made of these localities proves that ⅓ mile is ample width of channel for carrying the greatest flood, and indicates that one-third of the length of the magnificent, German-built bridge at the Tientsin-Pukow Railroad, at Lo-kou, would have sufficed, with great saving of cost if accompanied by some adequate simple river-training works.

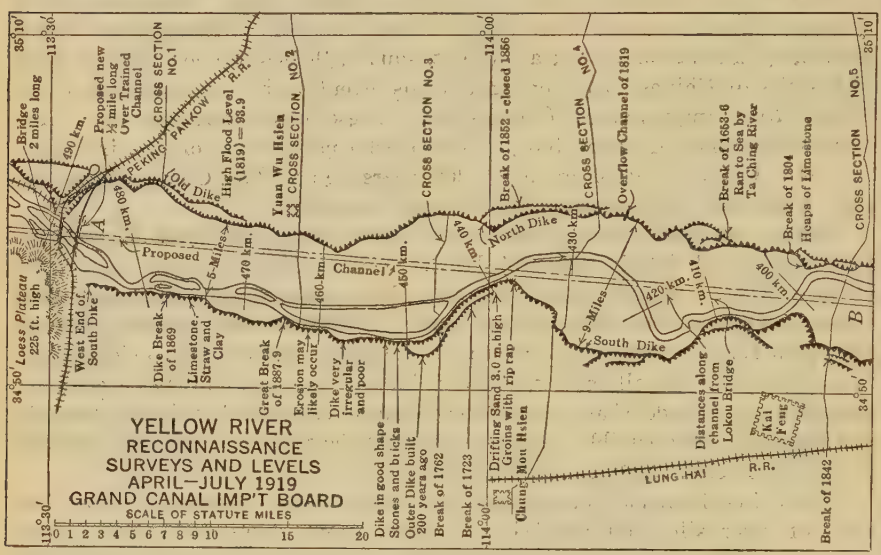


FIG. 6.

This great excess of width between dikes beyond that necessary for the flood channel or, for example, 5 miles instead of ⅓ mile, gives space for an ever-changing meander between these widely separated dikes, and when the bank is threatened by erosion, this space permits the river to be deflected away from the dike on a long new course by means of new spur-dikes. This method of obtaining safety, while convenient, is extremely wasteful of land. The speaker believes that a great saving of land is possible and that, by methods to be described later, the safety against the outbreak of a flood can be greatly increased.

In 1919, the Grand Canal Improvement Board, in the course of investigations supervised by the speaker, surveyed and ran levels for several hundred miles along the Yellow River dikes and leveled on cross-sections across the channel between the dikes, and extended these levels several miles across the plain outside the dikes, so that positive information at many places is now

available as to the relative elevations of river surface and of the ground outside, and, also, the slope of the river in flood and in drought is known.

In December, 1919, the speaker cruised, in a small boat, many miles down the river from near the apex of the delta, studying the dikes, currents, channel, sand-bar formations, and sites of some of the most famous dike

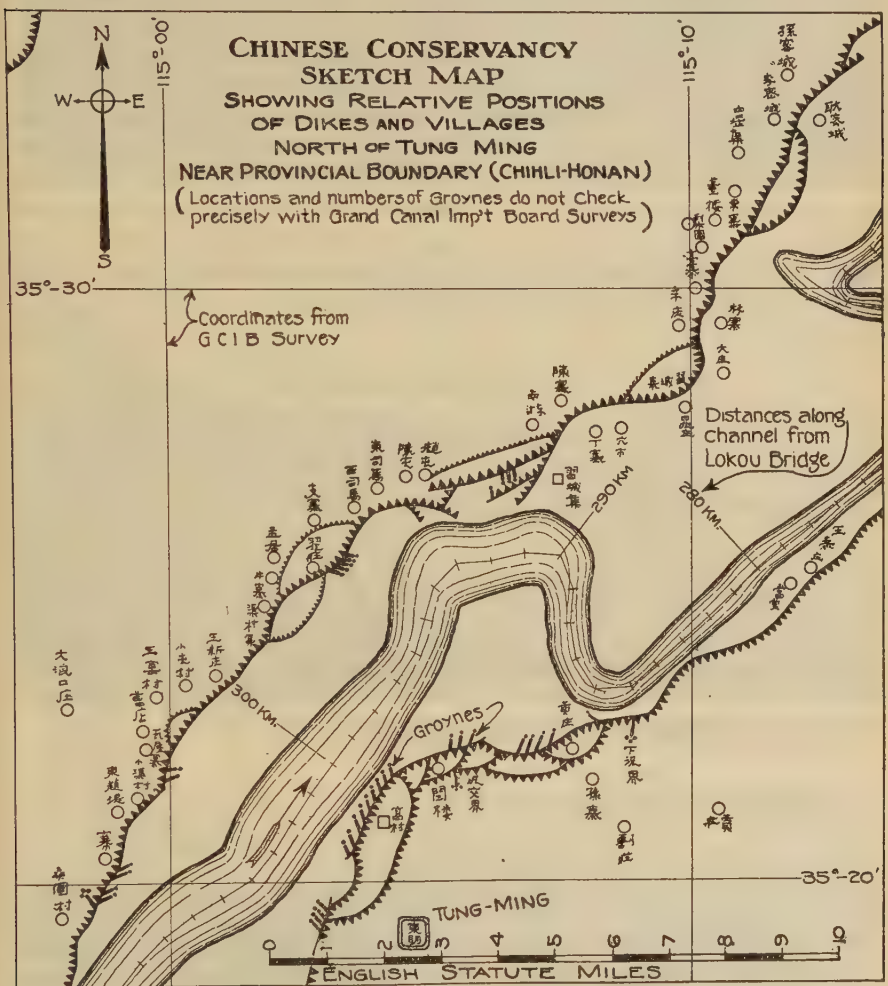


FIG. 7.

breaks, particularly those of 1869, 1887, and 1851. There seems to be hardly a 5-mile stretch of dike that has not been breached within the last few centuries, if the loops and groynes may be taken as evidence.

This excessive width between dikes prevails mostly within the up-stream 215 miles of the river's course within the delta. In the new course down near the old and new Grand Canal crossing, the inner dikes are little more than 1 mile apart for a distance of 20 miles along the river, but, here, the bed

is not elevated above the plain outside the dikes. Seventy miles farther down stream, below the Tientsin-Pukow Railroad Bridge, the inner dikes average only about 1¼ miles apart for the first 20 miles below the bridge and about 2 miles apart for the next 50 miles, according to the map in Capt. Tyler's report of 1905.

An example of the relations of channel, main dike, loop-dike, and spur-dike, is shown in Fig. 7, which was traced from a Chinese conservancy map of the part of the river near the boundary between Chihli and Shantung Provinces, and begins about 60 miles down stream from the end of the section shown in Fig. 8. It is reproduced because it is typical of many miles of dikes. The sites of threatened or actual breaks, more or less ancient, are shown by loops of adjacent dikes built either as a part of the scheme of regaining control or as a precaution at a threatened break. Records of attempts of the river to undermine and cut through the dike are shown by the succession of stone groynes or spur-dikes which were constructed, more or less hastily, to deflect the river from the threatened dike.

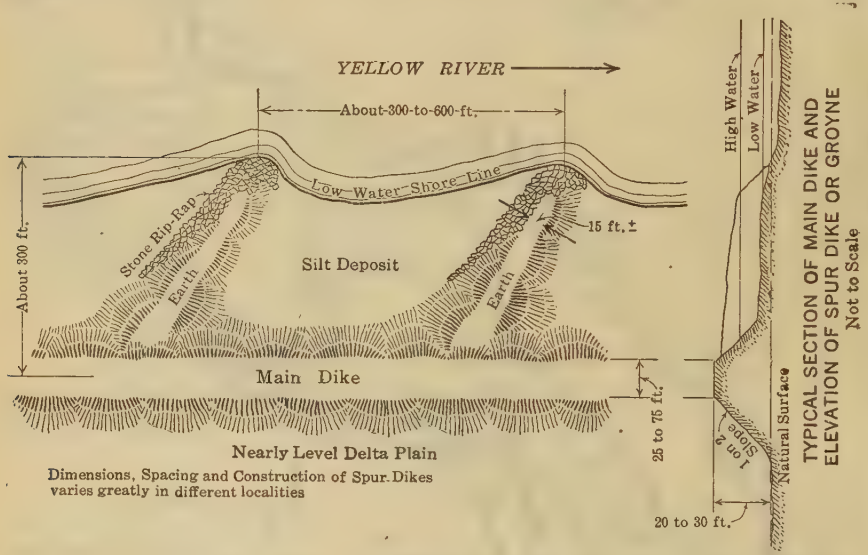


FIG. 8.

The general form and dimensions of these Yellow River dikes are shown in the photographs, Figs. 9 and 10. Fig. 9 shows the outer side of the main south dike at the site of the break of 1887, described subsequently. Fig. 10 is a view of a series of spur-dikes, looking up stream. Fig. 11 is a view of a dike gullied by rain, showing the friable nature of the earth.

Plate XIV and Fig. 12 show several cross-sections surveyed in 1919 approximately at right angles to the river's course, in order to find the relative elevation of the river and that of the broad plain outside the dikes, and serve to explain the tendency of the Yellow River to break out, cause terrible floods, and change its course across the river bed. These sections give the first

FIG. 9.—OUTER SIDE OF MAIN SOUTH DIKE AT SITE OF BREAK OF 1887.

FIG. 10.—SERIES OF SPUR-DIKES, LOOKING UP STREAM.

FIG. 11.—VIEW OF DIKE GULLIED BY RAIN.

published accurate instrumental levels of wide range across the dikes and the outside plain of the Yellow River. The gross exaggeration of this scale, 1 000 vertical to 1 horizontal, should be kept in mind while examining these cross-sections. This magnification of vertical scale was required to make plain the differences of elevation in a country so extremely flat that to the

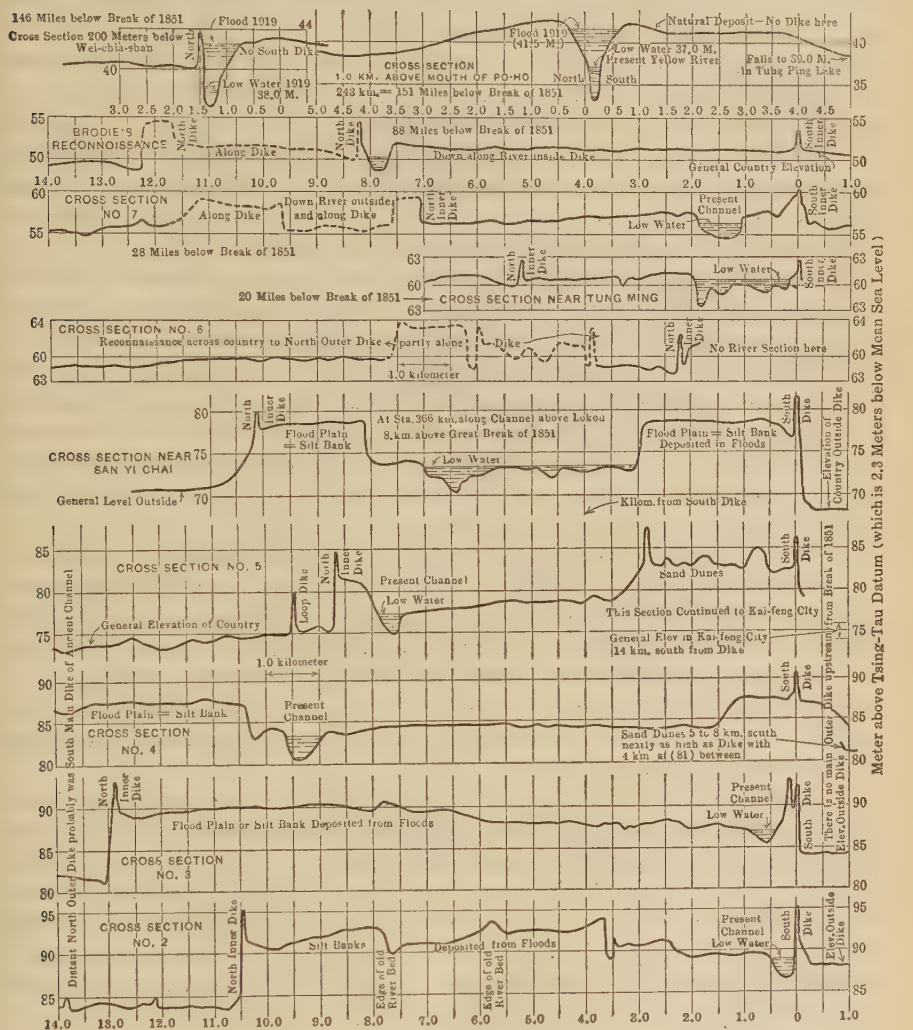

146 Miles below Break of 1851
Cross Section 200 Meters below Wei-chia-shan
Flood 1919
No South Dike
Low Water 1919 38.0 M.
CROSS SECTION 1.0 KM. ABOVE MOUTH OF PO-HO
243 km. = 151 Miles below Break of 1851
Flood 1919 (41.5 M.)
Natural Deposit—No Dike here
Low Water 37.0 M. Present Yellow River
Falls to 30.0 M. in Tung Ping Lake
BRODIE'S RECONNOISSANCE
Along Dike
88 Miles below Break of 1851
Down along River inside Dike
General Country Elevation
CROSS SECTION NO. 7
Down River outside and along Dike
Present Channel
Low Water
28 Miles below Break of 1851
20 Miles below Break of 1851
CROSS SECTION NEAR TUNG MING
CROSS SECTION NO. 6
Reconnaissance across country to North Outer Dike
partly along Dike
No River Section here
1.0 kilometer
CROSS SECTION NEAR SAN YI CHAI
General Level Outside
Flood Plain = Silt Bank
At Sta. 366 km. along Channel above Lokou
8 km. above Great Break of 1851
Flood Plain = Silt Bank Deposited in Floods
Low Water
Kilom. from South Dike
Elevation of Country Outside Dike
CROSS SECTION NO. 5
General Elevation of Country
Present Channel
Low Water
Sand Dunes
This Section Continued to Kai-feng City
General Elev. in Kai-feng City 14 km. south from Dike
1.0 kilometer
South Main Dike of Ancient Channel
Flood Plain = Silt Bank
CROSS SECTION NO. 4
Present Channel
Sand Dunes 5 to 8 km. south nearly as high as Dike with 4 km. at (81) between
Dike probably was
Flood Plain or Silt Bank Deposited from Floods
CROSS SECTION NO. 3
Present Channel
Low Water
There is no main Outer Dike upstream from Break of 1851
Elev. Outside Dike
CROSS SECTION NO. 2
Distant North Outer Dike
Silt Banks
Deposited from Floods
Edge of Old River Bed
Present Channel Low Water
Meter above Tsing-Tau Datum (which is 2.3 Meters below Mean Sea Level)

FIG. 12.

eye it appears a vast level plain. Plate XIV shows a greater width of the plain than Fig. 12 which is on a larger scale and shows sections above and below the break and change of course in 1851.

One of those cross-sections of special interest is that at San Yi Chai, Fig. 12, taken 5 miles above the great break, which shows plainly the super-

elevation of the river bed above the plain, and those at, respectively, 8 miles, 28 miles, and 88 miles, down stream from the break, which show no elevation of the river bed above the plain.

There has been much controversy among American engineers interested in the Mississippi levees about the super-elevation of the bed of the Yellow River above the outside plain, and some good men of high authority have denied its existence. The proof is now available. The facts are that for 50 miles or more in length, above the outbreak of 1851 and along the old course below this outbreak, the river bed is slightly above the plain, whereas along the new course, it is slightly below the plain.

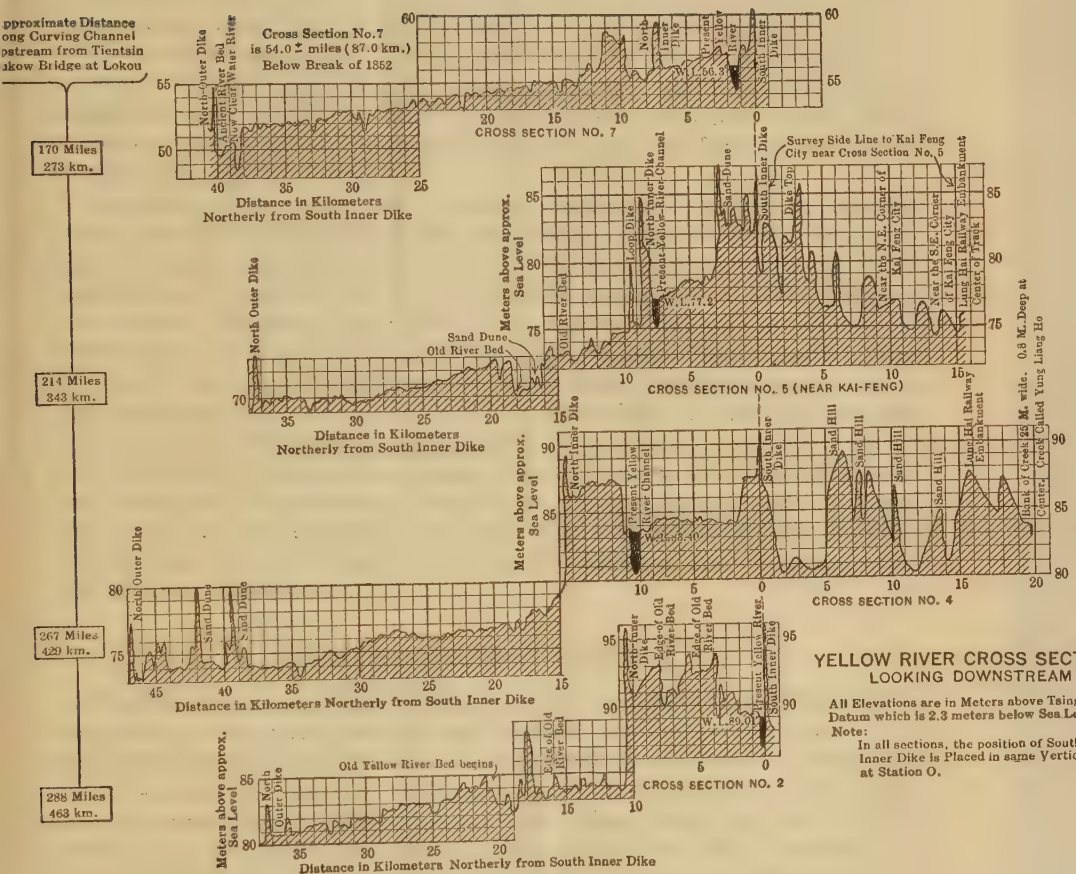
The three sections at the top of Fig. 12 are along the new course and those lower on the diagram are along the old course. All are drawn looking down stream and are arranged in regular order with the sections farthest down stream, or farthest east, at the top of the diagram.

These cross-sections surveyed in 1919 indicate that for about 80 miles of its length through the delta, or from the Peking-Hankow Railroad down to near the great break of 1851, the flood surface of the river is about 20 or 25 ft. higher than the level of the ground outside the inner dikes, and that the low-water surface is about 5 to 10 ft. higher than the ground outside the dikes; they also show that the normal bed of the river at low stages averages about 5 ft. higher than the general level of the country, this bed probably having been raised a few feet, by deposits of silt, to provide the needed slope for overcoming the friction of the increased length, as the river built its mouth farther out into the sea with the silt deposits of centuries.

Sections taken by the Kiang-huai Chinese surveyors along the abandoned channel of the Yellow River easterly from the Grand Canal in Kiang-su Province, in which the Yellow River flowed for 528 years until 1851, show that the super-elevation of the river here averaged about the same as that just described above the break of 1851, and thus curiously differs from the condition without super-elevation below the break of 1851, where the river has now been flowing for 70 years.

A longitudinal profile along the river surface and along the tops of both dikes also has been made up from this reconnaissance survey. This profile shows a remarkably uniform general slope of the river surface, except that there is a drop of about 10 ft. more than normal, within a few miles of the break of 1851, resulting in a largely increased slope for this distance. This is worthy of careful examination in the field, and why it has not cut back further and lessened this slope remains to be explained. Both the north and south river dikes are higher and thicker along the old course above the break of 1851, than along the new course below it.

In general, the accurate levels of 1919 confirm the statements of the English engineer, G. S. Morrison, and those of the Dutch engineers, Von Schermbeck and Visser, who visited the break of 1887 in 1888 and 1889, respectively, and reported the river bottom about 5 ft. above the level of the plain, and they disprove the statements of Gen. Wilson based on observations with only a hand-level, that no super-elevation of bed existed at the site of


pproximate Distance
ong Curving Channel
ostream from Tientsin
akow Bridge at Lokou
170 Miles
273 km.
214 Miles
343 km.
267 Miles
429 km.
288 Miles
463 km.
Cross Section No. 7
is 54.0 ± miles (87.0 km.)
Below Break of 1852
CROSS SECTION NO. 7
Distance in Kilometers
Northerly from South Inner Dike
Survey Side Line to Kai Feng
City near Cross Section No. 5
CROSS SECTION NO. 5 (NEAR KAI-FENG)
Sand Dune
Old River Bed
Old Yellow River Bed begins
CROSS SECTION NO. 4
CROSS SECTION NO. 2
Meters above approx. Sea Level
YELLOW RIVER CROSS SECTIONS
LOOKING DOWNSTREAM
All Elevations are in Meters above Tsing-Tau Datum which is 2.3 meters below Sea Level
Note:
In all sections, the position of South Inner Dike is Placed in same Vertical at Station O.

the break of 1851. None of these previous observers seems to have had the means for widely extended accurate leveling. Also, the speaker's reconnaissance disproved the old idea that there is a double set of good dikes, outer and inner, most of the way along the Yellow River, as a safeguard against the coursing of a flood far across the country, as in 1887.

Absence of Outer Dikes

Within these regions of super-elevation of river above the outside plain, which extend along the present course of the river for perhaps 80 miles down stream from the Peking-Hankow Railroad Bridge, greater danger is involved in the rupture of a dike, particularly on the south side where no outer line of dike exists; and it will be noted from Fig. 5, that most of the migrations have started on their new course from outbreaks in this region of super-elevation of bed. There is an outer line of dikes farther down stream, on the south side.

Along the north side of the river, near the apex of the delta, the so-called north outer dike of the present channel seems to be a south dike of a more ancient channel and below the break of 1851 the so-called south outer dike possibly may be one of the dikes of the river's course of 1194 to 1289; and this ancient abandoned river channel may have provided the site for an ancient canal described by the old records as existing in that vicinity. These outer dikes do not parallel closely the inner dikes, but are irregular and, in some localities, ten miles away from the inner dike. In general, the location or existence of this outer dike would seem at present to be more a matter of the accidental position of an ancient river course than of design for present-day protection. Nevertheless, these outer dikes sometimes serve a useful purpose in checking the course of a flood that has broken through the inner dike, and an example of such a checking and turning back of a minor flood will be given subsequently.

Yellow River Flood Discharge

The flood discharge of the Yellow River is from 200 000 to 300 000 sec-ft. and perhaps more, a few times in each century. In 1919, a flood higher than any of the preceding ten years, had a peak discharge of about 280 000 sec-ft., which is about the same as the maximum flood volume of the Mississippi above the confluence of the Missouri. The ordinary low-water flow is about 10 000 sec-ft. and continues ordinarily from October to May. Although the discharge from low water to high water thus increases twenty-five-fold, the mean velocity is increased only about two and one-half times. The necessary increased area of cross-section is obtained partly by the rising and spreading of the waters, but largely also by the river digging its bed deeper and wider, as will be described later.

This flood volume of the Yellow River is only about one-tenth of the flood volume of the neighboring Yang-tze River, but, nevertheless, it is a mighty flood and continues high from one to two months. The main flood of the year comes at any time from July to September, and is caused mostly by

rains or melting snows in the mountains and valleys, hundreds or thousands of miles up stream from the apex of the delta. Sometimes, brief important floods may come from heavy rains in the near-by mountains.

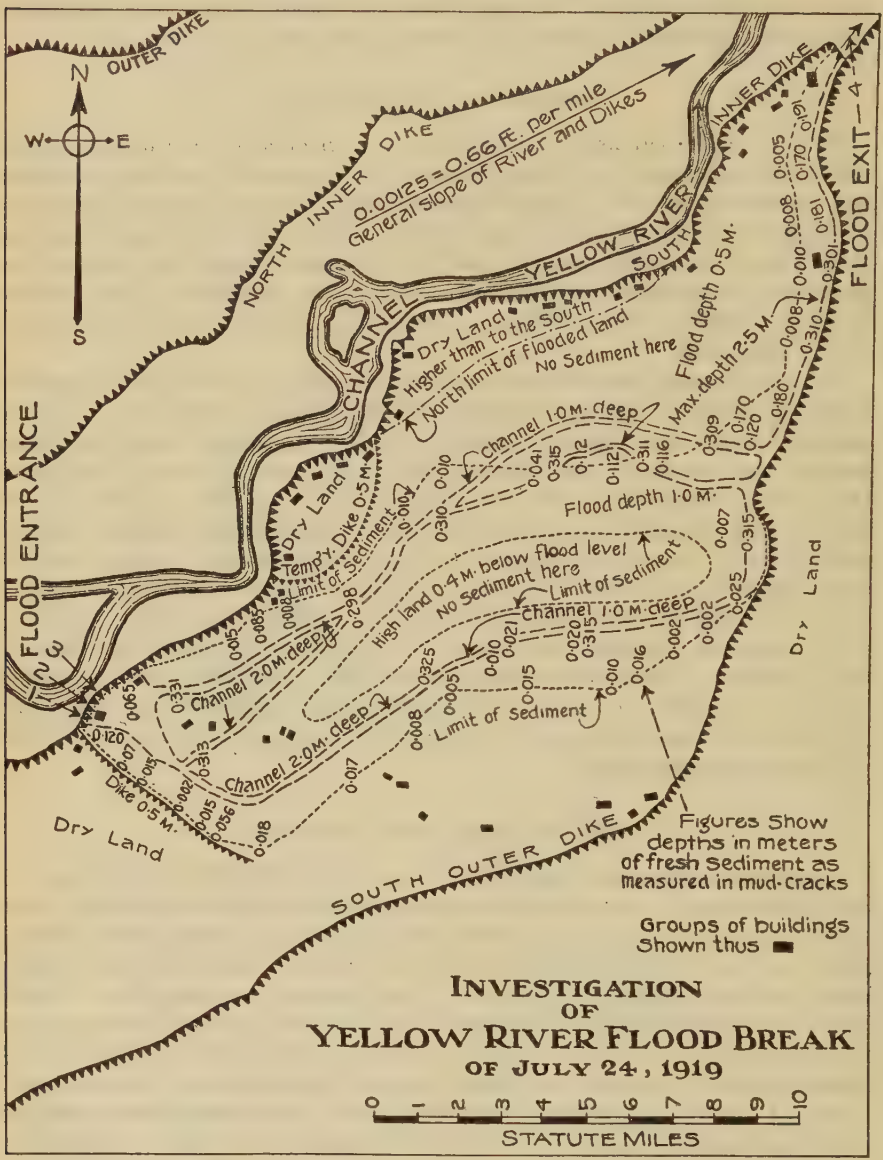


FIG. 13.

The log of a typical high flood, the highest of ten years, is given in Fig. 18. A good idea may be had of the variation from year to year in range, height, and duration, from the daily stage records, kept since about 1908

at the two railroad bridges, which show that the main flood may come at any time from July to September, but most commonly occurs in August, and continues for about five to eight weeks.

An isohyetal map of China (Fig. 25), is presented subsequently in this paper, in connection with Kiang-su problems; but rainfall studies give little aid on the flood problems of the Yellow River.

It is a remarkable fact that the Yellow River receives no tributaries within its long course of 400 miles through the delta, with the exception of the Ta-ching Ho, the bed of which it usurped in 1851. Obviously, they could not get in through the dikes. Moreover, here, as in most deltas, and as shown in Plate XIV, the river flows along a sort of crest, with the ground at right angles on both sides falling away on a slope, imperceptible to the eye, of only 6 in. to 1 ft. per mile. This outward slope of the flat delta cone obviously leads away from the river, the water pouring out through a break in the dike. This makes the control of the river, or the repair of a breach in a dike, particularly difficult, and gives to the Yellow River outbreaks, spreading sidewise down a delta cone, a character different from those of an ordinary river coursing down a valley.

When a flood makes a complete breach of a dike and escapes in large volume to the lower level of the ground outside, it quickly cuts a deep channel at the point of exit, which soon may become a gap a mile wide, and it is a serious matter to get the river back into its bed, particularly in localities where the bed is a few feet higher than the ground outside the dikes. Once in each few hundred years it has proved impossible to force the return of the river to its previous course, as in 1851-53. It came near to being impossible in 1887.

Remarkable ingenuity and skill, gained from centuries of experience, is shown in closing a large breach. In general, the method of the Chinese engineers, as described to the speaker by Messrs. Charles K. Edmunds and F. W. Tyler, the latter a retired sea captain who was for many years in the Chinese Coast Inspection Service of the Maritime Customs, is to narrow the breach by building out from the two ends with a current-resisting structure, skillfully made of crossed bundles of large millet stalks (Kao-liang) bound with bamboo or straw ropes, pinned down by small stakes, packed with river silt, and commonly weighted with more or less loose stone of about 1 cu. ft. in volume. Meanwhile, short spur-dikes at a slight angle down stream are built to deflect the current away from the new work. It is an extraordinary tribute to their skill that by the use of such weak and perishable materials as Kao-liang stalks and straw ropes, they can narrow the gap to about 100 yd. after being aided by the subsidence of the flood. This Kao-liang or giant millet has a stalk about 6 ft. tall and ¾ in. in diameter, with a thickly matted bunch of roots about 5 in. in diameter. It resembles the sorghum plant in shape and size. The bunch of roots is placed outward, against the impinging current.

For the final closure, many great ropes of bamboo fiber are stretched across the gap which finally is plugged either by lowering into it, or by floating into it, a sort of gigantic thick raft, skillfully built of great bundles

of millet stalks bound with bamboo ropes and held and guided by perhaps 100 bamboo cables, each nearly 3 in. in diameter, all constituting an exceedingly bold operation. As soon as the current has been thus checked, a thick reinforcing embankment of earth and stones is rapidly constructed, front and rear, by thousands of men with baskets of earth; which next is protected by stone rip-rap, and, meanwhile, the river is deflected away from the new embankment by the construction of spur-dikes.

Some Noteworthy Outbreaks

By way of further demonstration of the vast importance of finding a method of solving China's flood problems to the many millions of people inhabiting this vast delta between Kai-feng, Tientsin, and Chin-kiang (possibly more than the total population in the Buffalo-Boston-Washington triangle, for the great city populations in New York, Boston, Buffalo, etc., may be outnumbered in the density of China's farm population),* the following cases of serious floods from outbreak of the Yellow River are briefly stated. So recent is the opening up of that vast country to foreign engineering inspection and so imperfect have been the means of transmitting information, that the facts have been little known to the outside world.

The three most noteworthy outbreaks of the Yellow River in recent years of which details are available are as follows:

First.—The great break of 70 years ago, which changed the river's course for 270 miles, as shown in Fig. 5. This break occurred during the summer flood of 1851, but the water seems not to have been completely diverted into the new channel until 1853. The new course of this flood laid waste a stretch of prosperous country about 10 to 20 miles wide, and about 140 miles long, or nearly 2 000 sq. miles, and the escaping waters were not gathered into a narrow channel between dikes for this 140 miles below the break until about a quarter of a century had passed. No record is now known of the thousands of people who perished, or of the great value of the property ruined and the fertile farms destroyed along its path.

This diversion from the course that had been followed for 628 years also caused disaster to many thousands of farmers living along the deserted river bed by depriving them of their previous water supply.

Second.—The break of 35 years ago, which cost more than 1 000 000 lives and a vast property damage, came near causing a permanent diversion and was repaired after 1½ years of mighty efforts and great expenditure.

Third.—The break of 1903, about 13 miles below Tsi-nan (or "Chinan"), which is described by Capt. W. F. Tyler in his pamphlet† on the Yellow River. This outbreak deposited a bed of silt said to be from 3 to 8 ft. in thickness, over much of an inundated area of about 200 sq. miles, and caused such terrible distress as to call for organization of famine relief from foreign sources.

Details of the Great Break of 1851

It is said that foreigners first heard of this break 6 years after it occurred, and that 11 more years elapsed before anything definite about the cause of the

* The speaker has no precise data on this total delta population.

† Published in 1906, by the Maritime Customs Office at Shanghai.

river's change of course was known to the outside world. The first visit of a European to learn what had happened in 1851, was made 17 years after the catastrophe, by an English merchant of scientific education, residing at Shanghai, Mr. Ney Elias, who visited the new course and the scene of the break in October, 1868, and reported his findings to the Royal Geographic Society of England, which published his account.*

For many miles up stream from the site of the former course of the Grand Canal, he found the new river flowing irregularly over a strip of country about 10 or 12 miles in width, which had the appearance of a field, inundated and laid waste, rather than that of an ordinary river channel. The strip, thus laid waste, probably was 90 miles long. From 20 to 52 miles up stream, from the break, the river was flowing in a single, well-defined channel, where the shifting currents had deposited low, wide embankments of silt, perhaps 6 to 10 ft. deep, from which protruded half buried houses and temples. The river was flowing between the newly-made banks of silt deposit about 10 ft. high, with its bed at about the previous level of the country. Between the hills of Yu-shan and Chiau-kou, at the northwest corner of the Shantung Mountains, where the escaping waters found and usurped the ancient channel of the Ta-ching Ho, the waters were again gathered together and flowed to the sea, greatly swelling the volume of the Ta-ching River. Some reports state that the silt-laden water raised its deep-cut bed by deposits of silt; other observers, including Mr. Elias, reported that the added flow had cut the Ta-ching wider and deeper. Probably the discrepancy was due to different localities observed. Mr. Elias proceeded down the Ta-ching Ho to where it entered the sea to learn of its possible availability for navigation. The lower 20 miles was found to be flowing through low uninhabited mud flats, deposited from the river's burden of silt. The report by Mr. Elias is an admirably clear description which impresses one with the author's keenness as an observer.

Mr. Elias made a second visit for the purpose of exploring a new connection said to have been established in 1868, between the Yellow River and the Yang-tze *via* the Sha Ho and Huai River, through a break in the south dike 50 miles up stream from the City of Kai-feng. He found, however, that the breach had been repaired and that boat navigation was stopped early in 1870, after more than a year of flow through a breach 1 mile wide, but not deep. Mr. Elias, in his second paper, discusses the merits of turning the Yellow River back into the course it had followed prior to 1851, and calls attention to the great hardship that had been suffered by the large population along the old course of the river, in being deprived of water for irrigation and navigation. A large migration of this river thus brings disaster to millions of people along both the old and the new courses, because this fertile delta is populated in many places so densely that a farm of 3 to 5 acres has to provide the support of a family.

About ten years later, the new course was visited by a prominent English engineer, Mr. G. J. Morrison, who made an outline survey and map. The multitude of shallow shifting irregular streams flooding 10 to 15 miles in width, found by Mr. Elias, had then been gathered into a single stream between

* *Journal*, Royal Geographic Soc., Vol. XL (May 5th, 1871).

dikes, or had gathered itself, into this single, narrow channel by its erosion of its bed slightly below the general level of the plain as now found and shown in the top sections of Fig. 12.

An outline map of the region near the break of 1851, surveyed early in 1919, is given in Fig. 14. The speaker visited this locality in December, 1919, and found the landscape so broad and of such low relief that it was difficult for one in the field to take in the relation of one part to another. Although the chief events are plain, it will require a much closer and more detailed topographic map than is yet available, and a week in the field, map in hand, for tracing out in detail just what happened where the river changed its course. The river, breaking out to the north, found a country about 12 ft. lower in elevation than its former bed, over which it could flow, and over this, in ill-defined shifting channels, the flood ultimately reached the Valley of the Ta-ching Ho, within which, the records show, it had flowed about six centuries before, for at least about 80 miles, perhaps then reaching the sea by the Hsiao-ching Ho, instead of by its present course.

As stated previously, the escaping flood of 1851 usurped this Ta-ching channel and, in the course of 50 years, filled some miles in length of its bed with silt almost to the general level of the ancient delta plain, the depth of new silt thus deposited at Lo-kou being about 12 ft., according to borings made for the railroad bridge. Possibly, the "Ta-ching" or "clear-water" river which, although small, sometimes gives brief violent floods, by six centuries of effort had scoured this part of the channel bed below the elevation that it possessed when occupied by the Yellow River from 1194 to 1289.

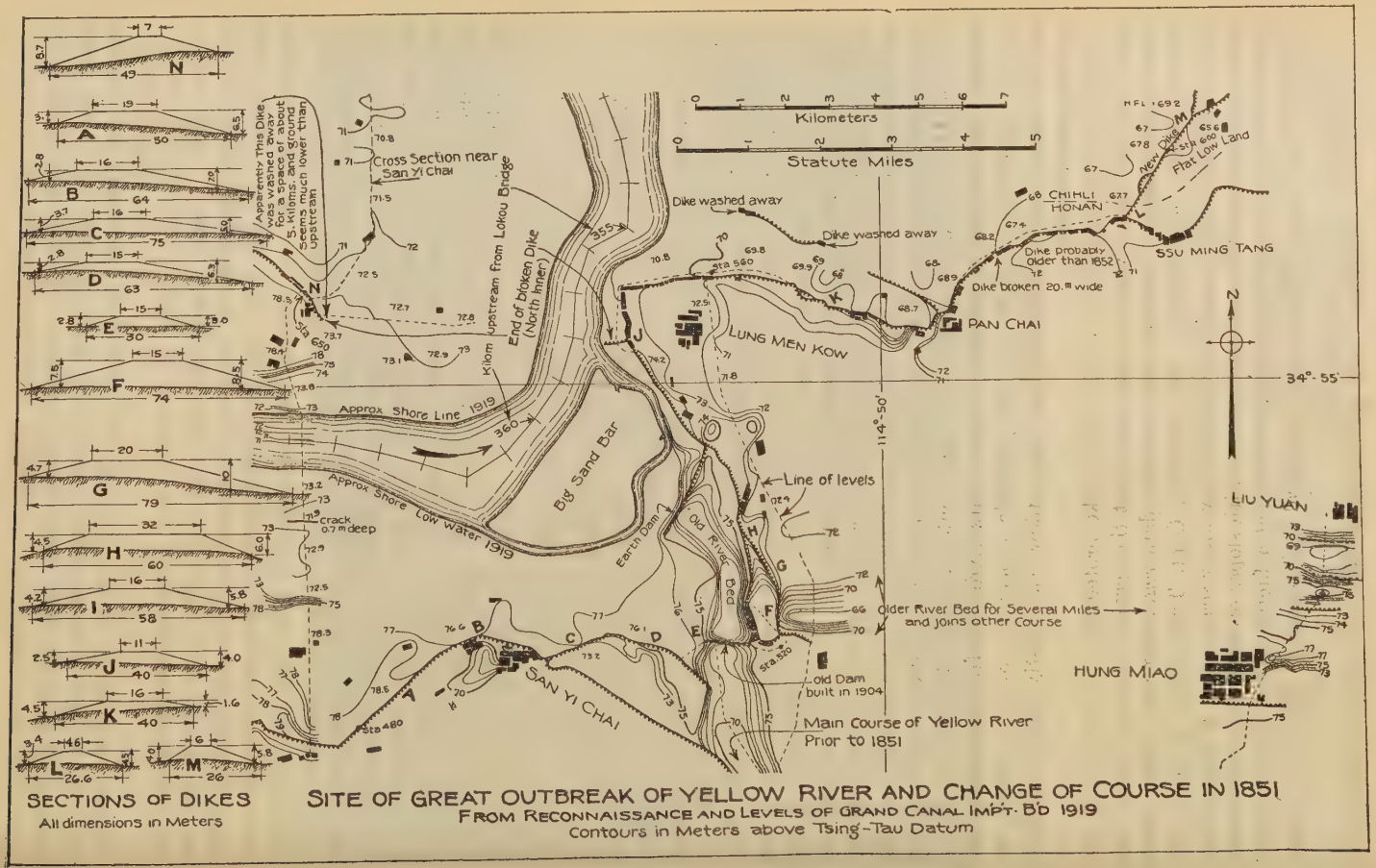
Some Details of the Flood of 1887

The outbreak of 1887, already mentioned, has been described by the English engineer, G. J. Morrison.* He says, "the breach through the dike was a full mile in width and the flood swept onward toward Hun-tze Lake and the Huai River, inundating a strip variously estimated at 20 to 50 miles in width, carrying away houses and villages and parts of walled cities."

Mr. Morrison reports that the breach occurred on September 29th, 1887, but was so great that it had not been closed when the next year's flood came in June, earlier than expected. All repair work of the previous months, which had cost a vast sum, was wasted, but the Chinese began resolutely again and, declining to follow foreign advice, adhered to their own ancient methods. At the time of his visit (September 1st, 1888), they had narrowed the gap to 400 ft. through which flowed an eddying current of about 7 miles per hour (10 ft. per sec.). He concluded that the original bed of the river was about 4 ft. higher than the level of the plain outside. The flood surface, of course, was much higher. His soundings showed a depth of 60 to 70 ft. in the breach and a deep gully for some distance inside and outside.

Mr. Morrison reports further, that the new banks were faced with Kao-liang stalks firmly packed in earth and that these millet stalks were brought, in some cases, by wheelbarrows from farms 20 miles away and that the earth for the new dike was all brought in baskets.

* *Engineering* (London), March 3d, 1893.



SECTIONS OF DIKES

All dimensions in Meters

SITE OF GREAT OUTBREAK OF YELLOW RIVER AND CHANGE OF COURSE IN 1851

FROM RECONNAISSANCE AND LEVELS OF GRAND CANAL IMP'T. B'D 1919

Contours in Meters above Tsing-Tau Datum

The final closure was made by a mattress of Kao-liang stalks and earth thoroughly bound together, suspended from many great bamboo fiber ropes across the breach, and lowered into place. The closure was completed on February 2d, 1889, and was greatly aided by a low stage of the river and by a strong ice cover.

Fig. 15 shows the interior of a temple that celebrates the closure of the break of 1887. Gordon Cumming* describes this flood as covering a strip 30 miles wide, having an area of 10 000 sq. miles, and flooding or destroying 3 000 large villages, with a probable loss of life of 7 000 000; and states that the flood continued high for two months and that the Chinese Imperial Treasury donated $2 500 000 for relief.

The speaker visited the site in December, 1919, and he has profound respect for the skill that under such difficulties could achieve success with such poor structural materials, such simple tools, and nothing but non-cohesive, fine sand with which to build this vast thick dike 30 ft. above the plain. Fig. 10 shows the site as the speaker found it, looking outside the dike across the gully of the flood.

Some Details of the Flood of 1902

Some features of the outbreak of 1902-03, on the lower river, are given in Capt. Tyler's report on the Yellow River, mentioned elsewhere in this paper, and some of the chief items of interest have been quoted previously. The breach occurred on September 12th, 1902, near the end of 2 months of flood and continued 6 months and 4 days. An attempt at closure on January 2d, 1903, soon after the flood had subsided, was frustrated by the capsizing of the mattress raft with which it was planned to close the final gap. Capt. Tyler states that in damage, maintenance, and expense of closing this breach, the Yellow River cost Shantung Province $2 000 000 in 1902. He attempted an estimate of the vast volume of silt deposited, after observing a few localities and seeking information from the natives regarding the other localities. He concluded that an average depth of 3 ft. over the 200 sq. miles might perhaps be true. A 3-ft. depth on 200 sq. miles would be about 400 000 acre-ft., a quantity so vast that the speaker cannot quite credit it after computing the probable discharge of water and its probable proportion of silt, carried at various stages, and considering that the outflow was mostly after the flood had largely subsided.

The colored illustrations of Kao-liang dike protection and groynes in the little book by Capt. Tyler,† are the finest that the speaker has ever seen. The book contains many interesting and valuable data regarding the Yellow River between Lo-kou and the sea. By sounding, Capt. Tyler found a maximum depth of 40 ft. in the channel pools in the low-water season and in some places a depth as great as 30 ft. against the bank, with slopes of 45%, of partly decomposed Kao-liang, to which he recommended that a facing of rip-rap be applied as providing the most efficient and economical maintenance. His chief recom-

* "The Leisure Hour", pub. about 1888.

† "Notes on the Yellow River", pub. by the Imperial Maritime Customs, Shanghai, 1906.

FIG. 15.—INTERIOR OF CHINESE TEMPLE WHICH COMMEMORATES CLOSURE OF BREAK OF FLOOD OF 1887.

mendation was that the low-water channel be straightened by groynes and by silting in the eddies below the groynes, and that the erosive power of the river, at all seasons, should be thus prevented from acting on the main dikes. He also suggested a system of temporary detention reservoirs along the river in the upper part of the delta, in which flood water might be detained and silt impounded for raising the general elevation of the ground.

Capt. Tyler states that he has recently found to be unreliable some of the information given him in 1902 as to the rate of the filling up of the Ta-ching Ho since the incursion of the Yellow River, which he quoted in this report of 1906. The actual filling is much less. He gives much other interesting general information of the Yellow River down stream from the crossing of the Tientsin-Pukow Railroad. About 30 years after the incursion of the Yellow River, and after the bed of the old Ta-ching Channel had become more or less raised, dikes were built from 3 to 7 miles apart, enclosing the enlarged river. Later, the riparian farmers, desiring to reclaim the land, were permitted to build, without much system or control, inner dikes only about a mile apart. These dikes, in the course of time, became the main dikes, and the outer dikes were neglected. The river floods now rise about 15 ft. above the general delta level and are restrained by the inner dikes.

Floods Along the New Course

Down stream from the outbreak of 1851, along the new course, as far as the confluence of the Ta-ching Ho, the surface of the Yellow River, when not in flood, is not elevated above the surrounding country, as has been previously stated and as is shown by sections at the top of Fig. 12. The outbreak of 1919, shown by Fig. 13, is of interest in illustrating the small danger of a permanent outbreak in that part of the river the bed of which is not elevated above the surrounding country, and in showing the efficacy of the outer dike. Nevertheless, this particular flood is reported by local magistrates to have spoiled the crops on 125 sq. miles, surrounded 560 small villages, drove out 217 000 people, and destroyed property worth $350 000.

In this case, the inner dike probably had been neglected, a gap having been worn in the road across the top was found by the rapidly rising flood which was higher than any of the preceding ten years. The outrush of water cut a deep channel at the point of exit and then spread in a broad thin sheet, flowing southward, ruining growing crops on the intensively cultivated little farms, until intercepted about 8 miles away by the south outer dike, along which it flowed about 20 miles parallel with the main stream, until caught in an angle between dikes, where, by reason of the inner, or river, dike being lower than the cross-dike, the flood broke its way back into the main channel.

Fig. 13 shows some typical channels, in which it appears that the water tends to gather itself by means of its power to erode the soil and carry it in suspension; but the general effect of these outbreaks is to spread a broad stratum of sediment over nearly all the country inundated.

A peculiarity of the Yellow River sediment is its remarkably rapid and almost complete subsidence in still or slow moving water, therefore this

stratum is thin under the pools farthest removed. Care was taken to measure the depths at many places and the results are shown in typical places on the map. In general, the depth over the local surface was about 1 in. and along the gullies about 1 ft. A rough estimate gave a total volume of about 12 000 acre-ft. of sediment deposited on about 60 000 acres. The Chinese engineer who made this reconnaissance, found difficulty in measuring the precise depth of sediment and, obviously, saw only a small part of the submerged 125 sq. miles in his rapid trip around the edges and down the gullies; the figures, therefore, on Fig. 13, are to be considered illustrative and not as a definite measure of the whole. The average depth was only one-fifteenth of that reported by Capt. Tyler from the outbreak of 1902, notwithstanding the water at the time of this 1919 break was carrying, by weight, about 4% of sediment; but here, in 1919, only a small portion, perhaps 20%, escaped from the main channel and this stopped flowing as soon as the river stage fell below the level of the country; whereas, in 1902, the whole river broke out, and continued out of its channel for six months. These two divergent instances are quoted to illustrate that one must not be hasty in generalizing about silt deposits from outbreaks until all the facts are known.

Ancient Records

A research among the city, provincial, and National records, probably would give a story of hundreds of outbreaks and show that at one place or another along this 400 miles of the Yellow River's course, for several thousand years, the population far and wide over this vast delta has lived in great danger, which danger exists to-day.

To illustrate the care with which the Chinese keep records in even the smaller towns: At the little Town of Lan-yi, near the great break of 1851, the speaker asked the magistrate about previous breaks, and a few hours later he had reviewed the town records for the preceding 200 years and presented a concise statement of about a dozen important floods or outbreaks in that immediate vicinity.

The Finje Report on the Yellow River

Because of the heavy floods in the Yellow River delta in the autumn of 1887, a reconnaissance was made in 1889 of the Yellow River through the delta, at the request of the Imperial Government, by two Dutch engineers, Von Schermbeck and Visser, and under the general counsel of Mr. Finje von Salverda, an eminent Dutch authority on rivers and harbors. Mr. Finje did not himself visit China, but wrote a sixty-page introduction, comprising many observations by Richthofen and sundry philosophical discussions. The interesting report by his agents adds little precise information to that given by Messrs. Elias and Morrison, but confirms their statements. Messrs. Visser and Von Schermbeck express opinions favorable to training the river, and state that a limiting of width might be advisable, noting that at Tsi Ho, a few miles above Lo-kou, the river got along very well in a channel only 1 770 ft. wide. They agree with

Mr. Morrison that, at the break of 1887, the bottom of the Yellow River originally was about 3 to 5 ft. higher than the ground outside the dikes. They record their observation, which is confirmed by the speaker's surveys, that "the whole of the new course from the great breach of 1852, appears to be below the general level of the country immediately adjacent to the river" and list sixteen outbreaks through the Yellow River dikes in 16 years in Shantung and Chihli, which affect Shantung Province.

Past and Present Safeguards against Yellow River Floods

From time immemorial, the maintenance of the Yellow River dikes has been one of the principal functions of the Government of each Province. Capt. Tyler states that until 1901 the whole river was in charge of one office, the River Viceroy, and that since then each Provincial Chief is responsible for the section of river within his Province. The conditions and methods described by Capt. Tyler doubtless prevail largely to-day. In Shantung, in 1903, he found three river Taotais controlling, respectively, the upper, middle, and lower thirds, and each Taotai had under him six military captains who were practically the resident engineers. Taotais and captains had been brought up from youth on this river work, and he met several who had been employed on it for 50 years and who remembered conditions along the Ta-ching Ho prior to the invasion of its bed by the Yellow River, earnest, careful men eager for new ideas, but extremely conservative in following ancient practice. Each captain had charge of only one side of the river. In protecting his bank, if he deflected the river against the opposite bank, that was the affair of the opposite captain. The speaker presumes that the care given under the Imperial System may have relaxed in some sections, with the disturbed political conditions of recent years.

In 1903,* Capt. Tyler estimated the expenditures on the Yellow River at 3 500 000 taels annually, equivalent to about $2 500 000. He estimated the annual destruction of property by the river's floods at about $1 000 000, without including the great catastrophes of 1851 and 1887.

Capt. Tyler's report gives an interesting account of current practice on the lower river down stream from the Tientsin-Pukow Railroad in 1903, showing forms of shore protection and a method of closing a break. The groynes shown by him are constructed from Kao-liang, or millet stalks, but their purpose is the same as that of the stone groyne shown in Fig. 8. The shore protection, that the natives call the "fish-scale" type, is built also of millet stalks, tied, pinned, and packed together with earth tamped into the interstices. The "fish-scale" or serrated form of Kao-liang bank protection is considered by Tyler as a deliberate design, in preference to a smooth face, and that this preference is based on experience with this perishable material which has a stalk about 6 ft. tall, that is weak and composed mostly of pith, but which has a mat of firm strong roots that are placed outward in the embankment to receive the impact of the current. In course of a year or two, the Kao-liang rots as the

* "Notes on the Yellow River", p. 10, Shanghai, 1906.

river rises and falls and a "pakwerk", 25 ft. high, may settle 4 to 8 ft. As long as the river persists in its attack on this particular piece of bank, the protection of Kao-liang is renewed by placing more on at the top. The mat of Kao-liang that is settled and compressed below the low-water level may last a long time. In many places, the Kao-liang groyne is faced later with a rip-rap of stones averaging 1 to 2 cu. ft. in volume.

No protection works face the earthen main dike, except where the river now impinges, or where, in former years, it has attempted to cut through.

The Yellow River Digs Deeply in Floods

The remarkable erosive action on the bed of the Yellow River and the vast quantity of the fine-grained silt that this river can carry in a day are "great forces of Nature" that the speaker would propose to "use for the benefit and convenience of man," thus fulfilling the traditions of Civil Engineering as defined by Tredgold in the charter of the Institution of Civil Engineers of Great Britain.

In the course of investigations on the channel of the Yellow River preliminary to designing its crossing by the Grand Canal, it has been found that, during a flood, this river digs its channel deeper and wider in a most remarkable way, and thereby gains at the bottom of the normal river bed much of the additional area of cross-section required by the enlarged volume.

As the flood subsides, silt is deposited, refilling the channel and raising the bed to its former elevation and contour. The speaker was on the lookout for this phenomenon, because years ago his attention had been called by Horace Ropes, M. Am. Soc. C. E., to a similar phenomenon in certain rivers of New Mexico and Arizona; also similar erosion and refill had been observed on the Colorado at Yuma, Ariz., which the speaker had visited and which at that point and below greatly resembles the Yellow River in its silt burden.

Similar erosion and refill also has been noted by English bridge engineers on certain rivers in India, but in none of these cases previously published have the observations been so extensive in following the changes through the rise and fall of a flood, or so carefully confirmed at every stage, as in these measurements of velocity, depth, and silt burden, made at four stations along the Yellow River, many miles apart.

This fact of the readiness of the Yellow River to scour a deeper channel when properly confined between dikes, taken in connection with the practically unlimited depth of the bed of silt in which it can dig, together with the hydraulic law of increase of velocity with increase of depth, makes it feasible to maintain a narrow channel at all stages from drought to flood, that can quickly adjust itself to carry a large volume of flood water. Before discussing the training of the river in a new straight channel, however, the proof of the facts about erosion and refill will be presented.

For frequently measuring the depth, width, velocity, and discharge of the Yellow River, gauging stations were established by the Grand Canal Improvement Board, in 1918-19, at the following places:

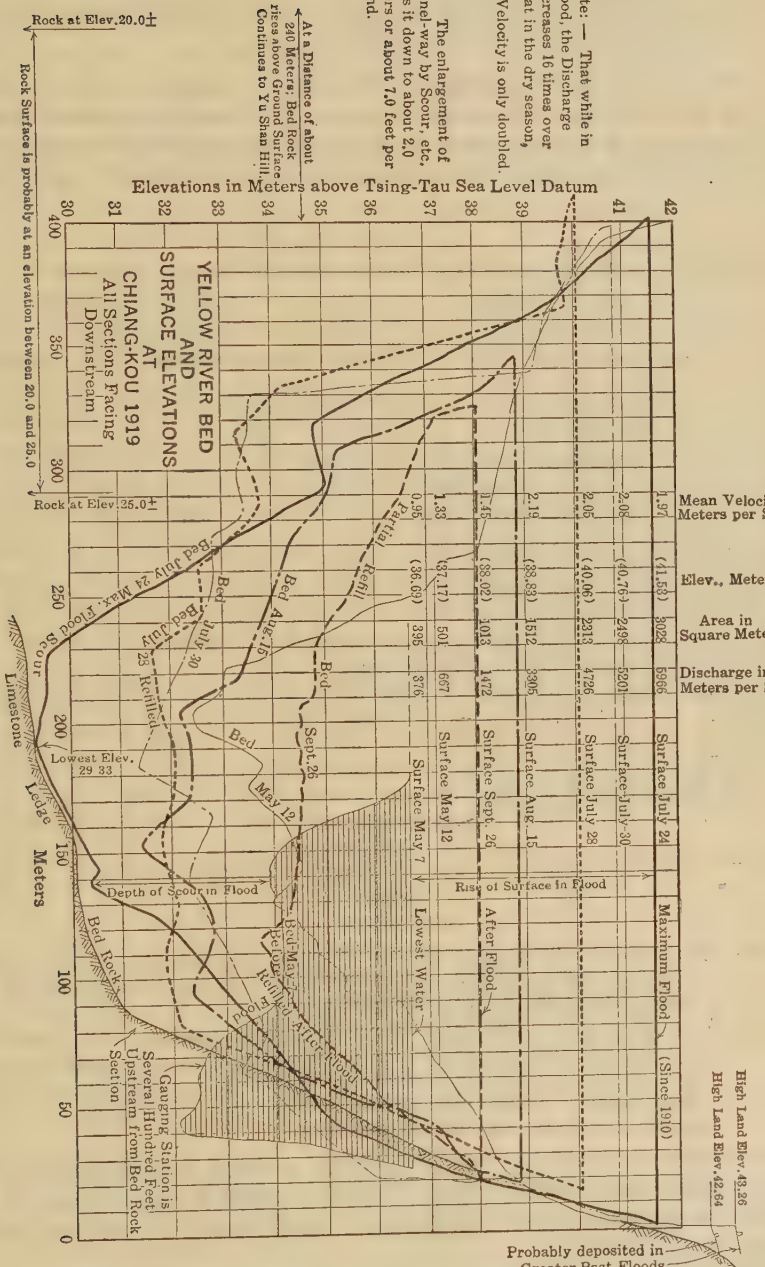

Note: — That while in Flood, the Discharge Increases 16 times over that in the dry season, The Velocity is only doubled.
The enlargement of Channel-way by Scour, etc. keeps it down to about 2.0 meters or about 7.0 feet per second.
At a Distance of about 240 Meters; Bed Rock rises above Ground Surface Continues to Yu Shan Hill
Rock at Elev. 20.0±
Rock Surface is probably at an elevation between 20.0 and 25.0
Rock at Elev. 25.0±
Elevations in Meters above Tsing-Tau Sea Level Datum
YELLOW RIVER BED AND SURFACE ELEVATIONS AT CHIANG-KOU 1919
All Sections Facing Downstream
Bed July 24 Max. Flood Scour
Bed July 30
Bed July 25 Refilled
Bed Aug. 15
Bed Sept. 25
Bed May 12
Refill
Lowest Elev. 29.33
Limestone Ledge
Depth of Scour in Flood
Bed Rock
Bed May 7 Refilled after Flood
Several Hundred Feet Upstream from Bed Rock Section
Gauging Station is
Rise of Surface in Flood
Surface May 7
Surface May 12
Surface Sept. 26
Surface Aug. 15
Surface July 28
Surface July 30
Surface July 24
Maximum Flood
(Since 1910)
After Flood
Lowest Water
Mean Velocity in Meters per Second
Elev., Meters
Area in Square Meters
Discharge in Cubic Meters per Second
1.97 2.05 2.08 2.19 1.45 1.33 0.95
(41.58) (40.06) (40.76) (38.89) (38.02) (37.17) (36.69)
3028 2313 2498 3512 1013 501 395
5966 4726 5201 3905 1472 667 376
High Land Elev. 43.26
High Land Elev. 42.64
Probably deposited in Greater Past Floods
Meters

FIG. 16

(1) Near the ancient canal line at Shih-chia-wa.
(2) At a possible site for a new crossing at Wei-chia-shan, 13 miles farther down stream.
(3) At another site for canal crossing, 6 miles farther down stream, at Chian-kou opposite to Yu-Shan.

At each of these places, systematic current-meter gaugings and soundings were made from the low-water stage in the spring through the various rising and falling stages of a flood greater than any of the preceding ten years and extending into the autumn when the river had nearly fallen to its normal stage.

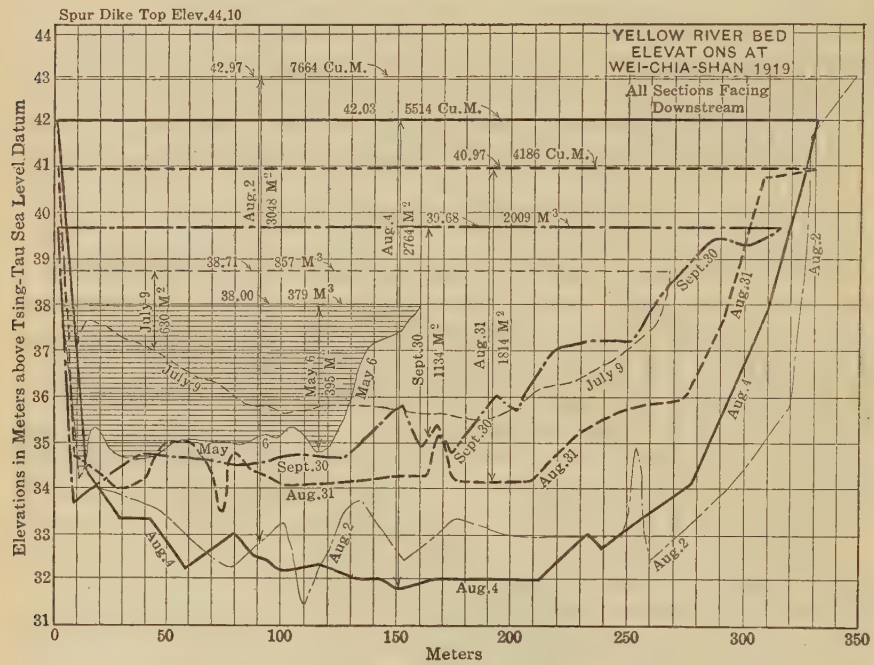


FIG. 17.

(4) Meanwhile, the Chihli River Commission established frequent gaugings at the bridge of the Tientsin-Pukow Railroad about 70 miles down stream from Chian-kou.
(5) A station, about 300 miles up stream from Station No. 1, within the hilly country up stream from the delta, was also established by the Chihli River Commission for obtaining a rating curve for a cross-section where the bottom of the river was supposed to be stable, hard, and not subject to scour.

These several cross-sections were purposely selected as having different characteristics of outline and velocity of approach, with a view to using each as a check on the other and to guard against being misled by abnormal conditions at any one location.

The results of soundings and gaugings at all four locations confirmed one another in a most satisfactory way. The more important results for two of

these stations are shown in Figs. 16 and 17, in which, for clearness, there have been plotted only a few typical cross-section lines for different stages. In every case, as the discharge increased, the river dug its bed enough deeper and wider so that, with the rise and spread at the top, the mean velocity of the whole cross-section never rose above about 2½ m. (or about 8 ft.) per sec. As the flood subsided, the space thus excavated was refilled with a most remarkable constancy of relation.

In many other gaugings of streams within the China delta plain, the speaker has found that the mean velocity of the entire cross-section seldom rises to more than 6 or 8 ft. per sec., in floods. In other words, given a material that can be eroded, the current digs out the bed and carries off the excavated material in suspension, until it has made for itself a comfortable channel in which it does not have to hurry.

Some careful tests were made for finding the truth about the oft-quoted statement that near the bottom of this and other streams heavily laden with silt, there is a flowing stream of soft, semi-fluid mud 1 ft. or more in thickness. The silt sampling by means of pump and pipe immersed at various depths showed nothing of that kind, but in order to be sure about it the speaker had special tests made at several points near mid-channel under the Lo-kou Bridge of the Tientsin-Pukow Railroad, with a horizontal flat iron disk, 14 in. in diameter, screwed to the bottom of a long vertical iron pipe about 1 in. in diameter, which reached above water and was used as a sounding rod, and manipulated from a boat held in position by ropes from the bridge. Within this iron pipe was a ½-in. steel rod sliding up and down, projecting above the top of the pipe, which could be thrust several feet into the river bottom by applying the weight of the observer. The observer could feel the consistency and hardness of the river bed plainly with the disk and rod, and many trials showed it about as firm and sharply defined in mid-channel as in depths of 2 or 3 ft. near the shore. The existence of a flowing bottom stratum of semi-fluid mud was disproved.

Soundings were also made with a thin wire line and flat-bottomed, boat-shaped lead weights, capable of combination to give 30, 60, 90, and 120 lb. weight in air. The elevation of the river bottom determined by the heaviest weight was not materially different from that determined by the lightest weight, or from the depth measured by the disk and stiff sounding rod; all of which confirms the speaker in the belief that the depths measured in Figs. 16 and 17 are dependable and that there was, in general, no serious error or excess from the bellying out of the sounding line in the swift flood current.

The silt content that can be carried in the water of the Yellow River within the delta region is found to be closely related to the mean velocity. As the velocity increases beyond, say, 2 or 3 ft. per sec., more silt is taken from the bed into suspension, but it is dropped promptly when the mean velocity decreases. This is illustrated in Fig. 18, in which are given lines showing the daily height of the water, the rate of discharge, the percentage of silt by weight, and the mean elevation of the river bed for a width of about 200 ft. near mid-channel, each of these lines being a composite made up from the measurements

at the three principal gauging stations, all of which were in excellent general agreement.

It will be noted that there is a lag of from 7 to 10 days from the discharge curve to the line showing the erosion and refill of the bed. It would be interesting to have similar curves from gauging stations located, respectively, near

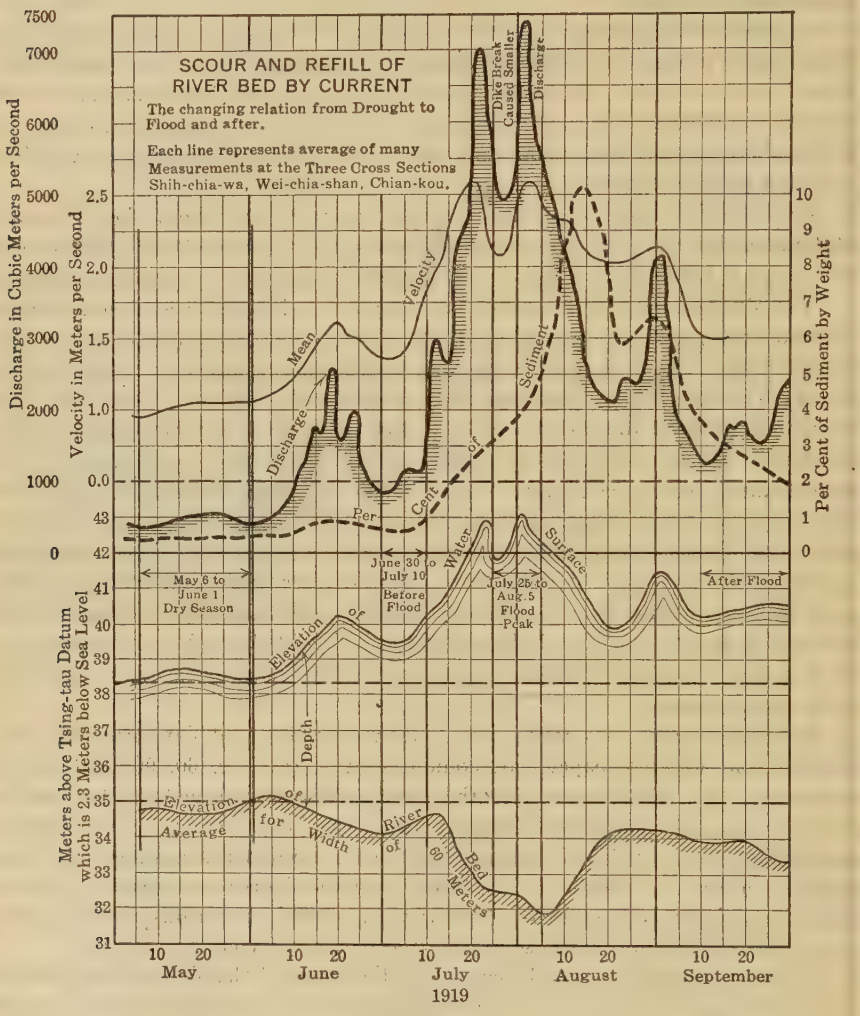

SCOUR AND REFILL OF RIVER BED BY CURRENT
The changing relation from Drought to Flood and after.
Each line represents average of many Measurements at the Three Cross Sections Shih-chia-wa, Wei-chia-shan, Chian-kou.
Dike Break Caused Smaller Discharge
Discharge in Cubic Meters per Second
Velocity in Meters per Second
Per Cent of Sediment by Weight
Mean Velocity
Discharge
Sediment
Per Cent of Water
Surface
May 6 to June 1 Dry Season
June 30 to July 10 Before Flood
July 25 to Aug. 5 Flood Peak
After Flood
Meters above Tsing-tau Datum which is 2.3 Meters below Sea Level
Elevation of Bed
Depth
Elevation of River Bed for Width of 60 Meters
Average
May
June
July
August
September
1919

FIG. 18.

the up-stream and near the down-stream end of the channel through the delta, or, say, 250 miles apart, for observing the influence of the quantity of silt taken into suspension from this erosion day by day, or week by week. All the speaker's observations tended to show that this silt picked up along the way forms only a small part of the entire silt burden brought down through

the delta from the erosion of the vast loess hills of Shansi and other Provinces far up stream.

A study of this phenomenon, excavation and refill of the river bed as velocity rises and falls, points the way to a successful training of this river by compelling it to dig and maintain its own flood channel and to carry its burden of silt to the sea. Many recent hydraulic investigations concur in showing that with increase of depth, velocity increases in a much larger ratio than that given by the Chezy formula which makes the increase proportional to the one-half power of the depth, whereas the newer formulas make it as the two-thirds or even the three-fourths power. This larger ratio is on the right side to be helpful in the present problem.

Characteristics of Yellow River Silt

The feature of special interest shown in Fig. 18, is the facility with which increased mean velocity was found to pick up an increased burden of silt and carry it along. The normal silt burden at the low-water stage averages about 0.4% by weight. After mean velocities of from 5 to 8 ft. per sec. had continued from 1 to 3 weeks, the silt burdens gradually increased to an average of 6.5% by weight, corresponding to 4.5% by volume as deposited on a compact bank. This 6.5% was found as the mean of eighteen samples taken at six widely different localities from July 31st to September 2d, 1919, during the high flood of the water; at the peak, the load was 9 or 10 per cent. The lag in picking up the full silt burden is very noticeable and indicates that a relatively small part of this is picked up locally from the bed, and that a large part may come from the action of the flood on the loess hills far up stream.

When one calculates these percentages of suspended silt, averaging upward of 5% for 40 days, into an average flood discharge of 150 000 sec-ft., the number of cubic yards of excavation transported per day or per month by the swiftly moving water, amounts to vast quantities in comparison with what man could afford to do by dredges or steam shovels. When one extends the computation to cover all the material transported by this entire flood of 1919, the volume becomes too great for one to believe that the silt movement always has gone on at this rate, year after year, through ten or twenty centuries. The shore line has moved seaward a long distance in the past 2 000 years, but hardly enough to cover such vast quantities. Pumpelly and Von Schermbeck and Visser cite a few localities and records, indicating a progress of the shore line seaward about 100 ft. per year, or about 1 mile in a century.

One should not generalize too far about the total annual volume of silt carried to the sea, until similar observations have been continued through the floods of several years. It is conceivable that somewhere in the loess hills of Shansi some particular cliff may have been cut into at an abnormal rate during this particular flood of 1919, or that something abnormal was going on during the outflow of 1902, which left the vast deposits described by Capt. Tyler.

Many hundreds of samples of this silt from the Yellow River and other streams along the Canal were collected, in 1919, at different flood stages, by subsidence from measured volumes of water and evaporation to dryness. Many other specimens were obtained by cutting out solid blocks from the river mar-

gins. The average of many specimens from many localities carefully collected and evaporated to dryness at 240° Fahr., showed for a densely compacted oven-dried cake a weight of 110 lb. per cu. ft. of final size as the maximum, and the ordinary oven-dried specimen weighed about 90 lb. per cu. ft. of original size as cut from the bank. Blocks with their natural water content as taken from a well settled bank along the river, averaged about 120 lb. per cu. ft.

In nearly all samples, the grains of this sand are of such microscopic size that 99% of the whole would go through a cement test sieve having 200 meshes per lin. in. The speaker examined many samples from widely distant localities, under a microscope with a power of 100 diameters. All these samples presented much the appearance of good mortar sand when thus magnified, and by measurement with an eyepiece micrometer, the diameter of nearly all the particles was less than 0.001 in., although a considerable percentage was only about one-tenth of this size. In spite of these extremely small dimensions, very little colloidal or adhesive quality was found, and nearly all the material in suspension in a glass of the turbid river water, 10 in. tall, would settle to the bottom within 15 min., leaving the water above with only a slight opalescence and so clear that fine print could be read through a column several inches in thickness.

This remarkably prompt subsidence has an important practical bearing on training the river so that material excavated in one place may be laid down in another place as desired by giving the silt-charged water a quiet pool in which to lay down its load.

This silt comes from vast deposits of loess in Shansi and other Provinces up stream, the most extensive in the world. In some places, these deposits are said to be more than 1 000 ft. thick. Where the speaker examined the loess hills, or the edge of the plateau, near the Peking-Hankow Railroad Bridge, the loess rises 225 ft. above the river, in steep bluffs, cut deep by small streams during heavy rains. *In situ* loess is a peculiar material and can stand in a bank with a vertical face many feet high; it cuts, smoothly, like cheese, absorbs water readily, and "dissolves" rapidly in a stream.

That sometimes the river carries sediment of much coarser grain, a true sand with grains averaging, say, $\frac{1}{10}$ in. in diameter, is shown by sand dunes outside the banks near the break of 1851, by dunes along the south dike up stream from the break of 1887, by vast fields of drifting sand along the Lung-Hai Railroad, and by drifts high against the eastern city wall of Kai-feng. Apparently, 99% of all the earth seen by the speaker along the Yellow River delta was of the fine-grain variety, of loess origin.

As the speaker cruised down the river in a small boat, he gave much attention to the ever-changing contour of sand-bars and shores and to the varying depths over the sand waves of the bed, frequently prodding the bed with a sounding pole and always finding the bottom yielding, but well defined. The standing waves in the surface of the water in many places gave evidence of sand waves, forming and slowly moving along the river bed. The low-water channel is extremely unstable.

Applications to River Training and Flood Relief

The preceding particulars have been given in some detail because of the possible wide application of the principles of river-bed erosion and re-fill to river training. From superficial observations in several other delta regions in China, the speaker is inclined to believe that a thorough investigation would reveal similar relations of velocity to scour of bed and its re-fill at several localities where flood channels are needed, and that the methods to be described for training the Yellow River could be made of wide application in China; but before laying out or beginning the construction of extensive works, he would, as a matter of course, insist on surveys, borings, test pits, and a variety of tests of physical qualities at the locality where these methods are to be tried, all made with great care and under competent direction.

One cannot yet be sure that all this silt of the great northeastern delta plain of China presents these characteristics found along the present course of the Yellow River between Shi-Chia-Wa and Lo-kou. Moreover, the speaker has read reports of investigations of silt from beds of smaller streams near Shanghai, which at moderate depths seem to present more of adhesive quality in the river bed and which might resist erosion.

The colloidal qualities of river-bed clays and silts and the causes of remarkable differences in adhesion, plasticity, and colloidal suspension are matters within the profound depths of molecular physics, which are only now beginning to receive the investigation that is merited by their practical importance in engineering operations.

A River-Training Laboratory

At present, there is only one laboratory in the world, as far as the speaker knows, designed for the special study of river-training problems. This is the Fluss-Bau Laboratorium at the Dresden Polytechnikum. In 1913, the speaker visited the old and the new laboratories at Dresden, and Professor Engels who is in charge, has courteously sent him accounts of some of his recent experiments therein, illustrating the formation of sand-bars and pools and the effect of groynes of various shapes and inclinations on scour and deposit at various velocities. Apparatus of this kind, but on a larger scale, is needed, both in America and in China, for a wide range of experiments on river training, and the speaker has told some of the Chinese statesmen that in skillful hands it might pay dividends of 1 000% per year on its cost, in their river and harbor problems. He would recommend working out the best means of channel training and the best shape of groyne for the Yellow River by trial-and-error methods, conducting experiments on a full-sized structure in the river almost simultaneously with experiments on the laboratory model, and thus seek to develop the precise form of the improvements in a tentative way, step by step; but he would follow the ancient Chinese practice of dike and groyne as far as possible, until it appeared certain that better methods and types had been developed.

A program could readily be planned, under which this careful proving up on designs during the necessary time for surveys need not delay the prompt and rapid carrying out of the sorely needed works of flood protection or flood prevention, along the Yellow River, the Wei or Hai River, and the Hwai River, or wherever funds are available.

Protection Against Floods

A Possible Method of River Training

The chief object in the river training now proposed is protection against floods by ultimately forming a thick, flood-proof dike by means of silting up the space between the present inner dike and a new straight dike built to confine the new straight and narrow channel and hold this from meandering, so that it will henceforth flow everywhere between new banks protected against undercutting by spur-dikes, somewhat as shown in Fig. 19.

All things considered, the Chinese have done wonderfully well in their river training by the methods they have followed for hundreds, and perhaps thousands, of years; but it should be possible to make great improvements by means of precise scientific observation and by slow painstaking study in laboratory and field with modern instruments and methods. The facts already observed have convinced the speaker that it is possible to utilize some of these marvellous forces of Nature found in the erosion and transportation of silt, to perform much of the labor of building and maintaining the channels and embankments needed for protection against the outbreak of floods.

In illustrating what the speaker now has in mind, he desires first to make it plain that the forms and dimensions shown in the following drawings may need to be modified after further observation and study in the field and study in a hydraulic laboratory. With this reservation, the speaker presents the outline design, shown in Fig. 20.

The river would be compelled to flow in straight courses, 5, 10, or 20 miles in length, laid out nearly midway between the present inner dikes, as may best fit existing conditions in different localities, somewhat as shown in Fig. 6 by the parallel straight lines.

The new flood channel would be made only about $\frac{1}{3}$ mile in width at the surface, the centers of the new main dikes being less than $\frac{1}{2}$ mile apart, instead of from 4 to 8 miles, as at present.

Various means in different localities would be used to swing the river into the new straight channel, utilizing the natural erosive power of the current to the greatest practicable extent and building out long spur-dikes to turn the current toward the new channel the opposite dike of which would have been previously built and provided with groynes, lest the current overshoot the mark.

The space of from 2 to 3 miles in width thus left between the old inner dike and the new straight dike, as shown in Fig. 19, would ultimately become silted up to nearly the level of ordinary floods, forming the most fertile kind of agricultural land, like the rich bottom-lands, or intervales, of American rivers,

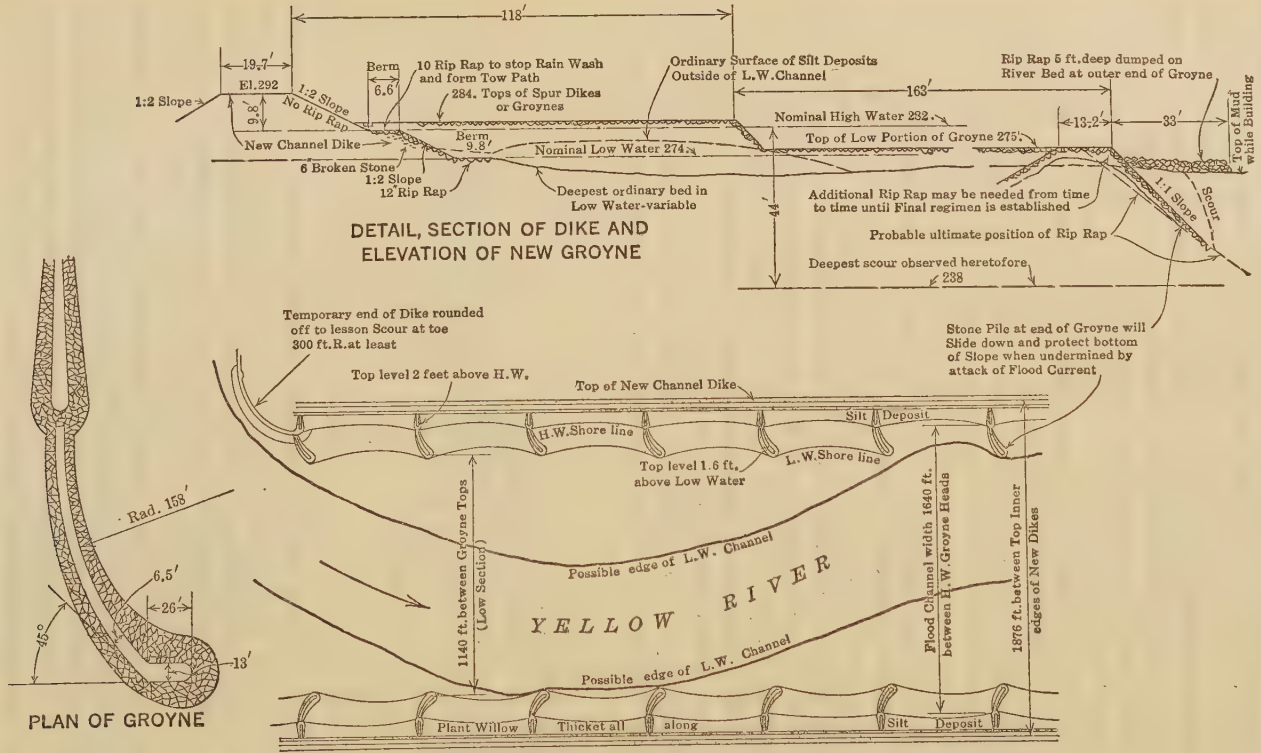

118′
19.7′
El.292
1:2 Slope
1:2 Slope
No Rip Rap
9.8′
Berm
6.6′
10 Rip Rap to stop Rain Wash and form Tow Path
284. Tops of Spur Dikes or Groynes
Ordinary Surface of Silt Deposits Outside of L.W.Channel
163′
Rip Rap 5 ft.deep dumped on River Bed at outer end of Groyne
Nominal High Water 282.
Top of Low Portion of Groyne 275.
13.2′
33′
Top of Mud Wattle Building
New Channel Dike
Berm
9.8′
Nominal Low Water 274.
6 Broken Stone
1:2 Slope
12′ Rip Rap
Deepest ordinary bed in Low Water-variable
Additional Rip Rap may be needed from time to time until Final regimen is established
1:1 Slope
Scour
44′
Probable ultimate position of Rip Rap
Deepest scour observed heretofore
238
DETAIL, SECTION OF DIKE AND ELEVATION OF NEW GROYNE
Temporary end of Dike rounded off to lesson Scour at toe 300 ft.R.at least
Stone Pile at end of Groyne will Slide down and protect bottom of Slope when undermined by attack of Flood Current
Top level 2 feet above H.W.
Top of New Channel Dike
H.W.Shore line
Silt Deposit
Top level 1.6 ft. above Low Water
L.W.Shore line
Rad. 158′
6.5′
26′
45°
13′
1140 ft.between Groyne Tops (Low Section)
Possible edge of L.W. Channel
YELLOW RIVER
Flood Channel width 1640 ft. between H.W.Groyne Heads
1876 ft.between Top Inner edges of New Dikes
Possible edge of L.W. Channel
Plant Willow
Thicket all along
Silt Deposit
PLAN OF GROYNE
ARRANGEMENT OF DIKES AND GROYNES FOR STRAIGHT NARROW CHANNEL
(Subject to revision after observation in Field and experiment in Laboratory)

FIG. 19.*

* It may be found impracticable to build the outer part of the groyne to less than flood height, because of scour from the swift current running over its top and following along the down-stream slope and undermining its base; but, if practicable, the lower groyne is

but presenting the great superiority of having the admission of flood water always controlled, until growing crops had been harvested, by substantial sluiceways and gates built into the new dikes. After the crop had been harvested, the occasional admission of more or less flood water would add to the fertility and aid the maintenance of a deep-seated reserve of moisture in the subsoil. It is obvious that when this broad terrace of earth between the old dike and the new dike has become built up to about the level of ordinary floods, if subsequently the inner dike becomes overtopped by an unprecedentedly high flood or undercut by the current under extraordinary conditions, this barrier of elevated ground about 2 miles in width in front of the old dike, would prevent any rapid outrush of water from causing disasters like those of 1887 and 1851, or the countless series of outbreaks that have occurred from time immemorial all along the course of the river through the delta.

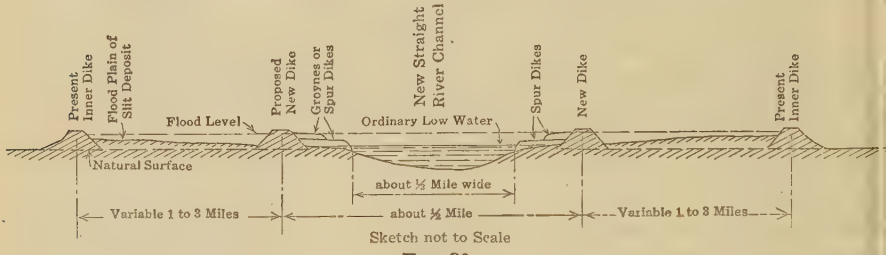


FIG. 20.

The speaker would make each necessary change of direction between these long straight reaches of new channel by means of a relatively short sharp curve of a radius only two or three times the channel width. At present, he attaches little importance to the theory often mentioned in papers on river training that because rivers naturally flow on curves and not on straight lines, therefore, a gently curving course is the best in which to train a river. Nature's chief object in the winding course of rivers seems to be delta building and the distribution of sediments eroded and brought down from the hills; whereas, on the contrary, the object of man in this case and many others is to prevent meandering and bank-cutting and to cause the river to carry its burden of silt in the flood water to the sea in the most direct and certain way possible. It is not believed that the interests of future navigation of the Yellow River would be sacrificed by these straight courses, or that, as a whole, the river dikes would be more difficult to maintain.

Although flood protection is the controlling motive, navigation must not be forgotten. At present, the navigation of the Yellow River is confined to a multitude of small boats of about 3 or 4-ft. draft, 10 ft. wide, and about 40 to 50 ft. long. The shoals are many and capricious. It seems certain that the new, confined and deepened channel would bring great improvement over present conditions.

As to possible troubles from hills and hollows, or shoals and pools, in the river bottom, caused by the alternate obstruction at the groynes and the enlargement and eddying below the groynes, the speaker was encouraged to

believe that a satisfactory condition could be worked out, by his observations of the currents, shoals, and eddies at the old Chinese groynes along this river, during his cruise between the railway bridges in December, 1919. Nevertheless, he would propose a careful study of the best spacing for groynes by laboratory models, before building many structures in the field, would take long and careful counsel from the most experienced Chinese conservancy superintendents, and would carefully study the sand waves in the bed, of which he noted many examples as he cruised down the river.

The long continued and almost general use of the spur-dike by the Chinese for turning an impinging current away from a dike built of soft, easily eroded earth naturally calls attention to that type of construction as having been proved by centuries of experience to be well adapted for meeting local conditions. Moreover, it takes little arithmetic to show that a smaller quantity of stone rip-rap will be required for protecting a mile of river bank when this is carefully and scientifically disposed in spur-dikes than if placed in a continuous line of rip-rap parallel with the shore. Also, any sudden freak of the river in threatening the dikes at a particular point can be most quickly warded off, with a smaller quantity of stone and by fewer men, when the material is dumped off the end of a narrow spur-dike.

In brief, although the speaker has confidence in the general method herein proposed, he would at first build only 2 or 3 miles of new channel with the new shape of dike and groyne, and study its behavior and expect to improve the details. Meanwhile, he would seek maximum economy through painstaking observation, experiment, and the counsel of others, and then proceed with greater lengths.

One of the most important problems for present consideration is that of devising the cheapest effective means of placing and holding the river in the new pre-determined course while making use of cheap materials near at hand. Along the Yellow River, and at convenient intervals on many of the Chinese delta streams, limestone hills project through the vast deep deposits of silt, from which any desired quantity of stone for rip-rap can be quarried. In some localities, the river flows at the foot of these hills, making transportation by native boats economical; this fact suggests the use of rip-rap much as the Chinese have used it in their spur-dikes from time immemorial.

It is quite within the range of probability that the straightening, shortening, and deepening caused by the new system of a straight and narrow way would gradually cut down the bed of the river to an elevation materially below the plain, so that the great danger which comes from silt deposits raising the bed and from the super-elevation of the normal surface above the general level of the country outside the outer dike would gradually become lessened. As to the possibility of trouble from interference with Nature's process of spreading silt from the Shansi hills over the delta surface, through arresting the occasional change of the river's course, or through the more rapid extension of coast line or the shoaling of the Yellow Sea, these possibilities are about as remote in the future as the Cro-Magnon civilization is remote in the past, and present generations need not worry.

Brushwood Retards.—There are many places where the cheapest and quickest means of diverting or of holding the Yellow River to a new channel would be by checking the current and causing sand-bars to form by means of a floating raft of bushy trees, if trees could be had. This part of China is practically treeless, and each bit of wood is so treasured by the Chinese peasant that it might require an army of soldiers to protect a quick-growing forest of cottonwoods or the like. Nevertheless, taking a long view of China's great problems of flood protection and river regulation, the Government might well consider immediately setting apart and planting suitable trees for this purpose. Trees of serviceable size for these purposes could be grown long before China will have its river-protection works one-tenth finished.

Training the Yang-tze

This river greatly needs training for preventing the destruction of valuable fertile shores by erosion and caving; also, for purposes of navigation in increase of depth, and a more permanent channel. For the Yang-tze River, with its far greater volume and depth, naturally there would be devised a different arrangement from that found to be most economical on the Yellow River. Along the Yellow River, thus far, no evidence has been found of deep holes being dug by the swirl of the current, to a depth of more than 30 or 40 ft. below the low-water surface, although possibly this may occur; but along the Yang-tze River, in its 600-mile course from the great City of Hankow (the "St. Louis" of China) to the sea, the navigation charts show many submerged gullies more than 100 ft. and some even 125 ft. in depth. Even when a current of this great depth attacks a protected shore line, defense would seem to be possible by some such special means as trees with bushy tops, sunk by heavy stones tied to their stems and laid over one another like shingles. The quick subsidence of the Chinese river silt favors the rapid building up of great deposits in eddies or where the current is checked by such material as bushy tree tops.

In general, the channel will seek the line of least resistance, and in this delta region there appears to be an almost unlimited depth of easily eroded silt which can be cut at the bottom in mid-channel to any needed depth on any desired location; and it is of great importance that, in the beginning, the new straightened and improved channel be laid out on the best possible lines.

Apparatus and Methods.—The cheapness with which this soft earth can be excavated, moved, and put in place by native labor, also must be considered in planning great works for flood relief. Methods that would be followed in America with its high-priced labor and convenient great machine shops would be out of place in China where earth can be excavated on a large scale and moved by men with native shovels and baskets at a cost of less than 10 cents per cu. yd., and where a working force of 10 000 willing and industrious laborers could be quickly assembled along any given stretch of river, most of whom would be greatly benefited by the distribution of money. A drag-line scraper with a long boom would be useful here and there for work below

the water level, but, in general, funds expended for manual labor would do far more good to humanity, distributed among the farmers for miles around, than if put into expensive dredges, steam shovels, and construction-railway material manufactured in foreign lands.

Agricultural Improvements

Although "Flood Problems" is the title of this paper, and the chief object of this work proposed along the Yellow River would be the safe-guarding of the many millions of population on farms and in small cities within a few hundred miles north and south of its course, from any such outbreak of flood as that of 1887 or of 1851, there would be an important incidental benefit from converting the strip of land, from 4 to 8 miles in width, between the inner dikes from its present condition of partial and hazardous use to one of security, fertility, and maximum use, making it comparable with the Upper Valley of the Nile above Cairo. On this narrow strip, vast quantities of food could be grown with certainty, in years of greatest drought, in the midst of a land where fearful famines sometimes occur from the shortage of rain. Some of the worst famine districts, in which the American Red Cross and other humane agencies expended more than $1 000 000 in 1921, are close at hand to the strip of "river bottom" that could thus be made sure to yield two bountiful harvests each year, regardless of the general shortage of rainfall.

Regarding the agricultural development of the strip between the dikes, this would be subdivided into alternating tracts, the progressive development of which would be changed from time to time. For example, at first, there might be certain large areas outside the inner dike, set apart by low dikes, to be irrigated by gravity flow through carefully constructed conduits. The present super-elevation of the surface of the river above the level of this surrounding country could thus be made a blessing instead of a danger, since this super-elevation would provide the fall for distributing the irrigation water. To just what extent the abstraction of water for this purpose would be permissible remains for future consideration.

Another means of shaping the flood-relief work for aiding agriculture would be by establishing impounding reservoirs within cross-dikes between the old inner dike and the proposed new dike, each of which reservoirs might present several square miles of surface 10 or 15 ft. in depth of water until shoaled by silting.

These impounding reservoirs might be located on alternate sections and the reservoir section used for irrigating the section from which water was excluded by the new dike and after, perhaps, 10, 20, or 40 years, when this reservoir had become shoaled by silt, it could then be treated as agricultural and irrigated from the river by pumps of many kinds, some perhaps driven by wind-mills.

The Huai River Flood Problems

The flood region of the Huai River is shown in Fig. 3 and Fig. 21. The American Red Cross and others are said to have expended about $400 000* in 1911 in food, supplies, and other means of relief in the famine which resulted

* See Reinsch, "An American Diplomat in China", pp. 70–80.

from floods along the Lower Huai River, and followed this by a practical effort to help prevent such disasters in the future by sending an engineering commission composed of members of the Society to China to advise with the Chinese on means of improvement in this Huai River District.

Probably no place in the world more urgently needs improvement by works on a large scale. As has been stated previously, Mr. Chang Chien, scholar, statesman, and foremost Chinese industrial leader, moved by a desire to be of service to his native Province, had developed a school of conservancy engineers who had been collecting data and making surveys for several years prior to the coming of the engineering commission sent by the American Red Cross. The Red Cross engineers made a careful study of the data already collected by the Chinese, and under difficult conditions, made a somewhat hurried personal reconnaissance of the region. In the course of a few months, the Board submitted its report with recommendations for a plan of drainage channels and reclamation works, estimated to cost about $30 000 000; but no financing followed.

The problem appears to have been too large and too difficult for satisfactory solution with the data, the time, and the means then at the command of the Board; and its plan of works has not been approved by the Chinese, who have continued their surveys more or less actively under Chinese leadership. A year and a half ago (1920), His Excellency, Chang Chien, presented a tentative report for a scheme of relief radically different from that proposed by the Red Cross Board, calling for an expenditure, first and last, of about $45 000 000, at the present rate of exchange, suggesting that the work be done largely by the soldiers of the needlessly large Chinese Army which had been called into existence during the World War, and could not well be disbanded and turned loose without employment or means of support. Within the past year (1921), additional surveys have been going on and a young American engineer, E. W. Lane, Assoc. M. Am. Soc. C. E., recently of the Miami Conservancy and of the Morgan Engineering Companies, has been called to China to assist Mr. Chang Chien's organization on canal improvement and other hydraulic problems in Kiang-su Province.

When studied in a broad and thorough way, this problem of the Huai River flood relief is extremely complicated, combining river training, flood-detention reservoirs, irrigation supply, the exclusion of sea water from flowing back in the channels in time of drought, the improvement and modernizing of about 100 miles of the Grand Canal, as well as the reclamation for agriculture of vast areas of shallow lake beds. As previously stated, the finding of a practical solution intimately affects the lives and prosperity of 2 000 000 inhabitants in one of the most fertile regions of the world, which might be far more densely populated if only the present terrible succession of floods could be prevented.

Until about 728 years ago, affairs appear to have gone on comfortably in this district, as affairs go in China. Then, the Yellow River broke away from its older channel and usurped the bed of the Huai River and by raising its bed with deposits of silt, and thus finally completely excluding the Huai River from its ancient bed, forced the enormous Huai flood volume, sometimes

more than 250 000 sec-ft., to find new outlets to the sea, partly through scant sluice-ways in existing dikes and partly by way of the Grand Canal to the Yang-tze.

The Huai drainage basin comprises 52 000 sq. miles, on which sometimes 10 or 12 in. of rain falls in a single storm. The crude maps available indicate that more than three-quarters of the catchment area is hilly or mountainous, and that less than one-quarter is flat, low-lying delta bordering on the sea.

A few centuries prior to the forcing of the Huai River from its ancient bed to find new outlets, there had been built across the delta, a few miles back from the sea, the great Maritime Dike, 100 miles or more in length, for protection against tidal waves and pirates, and the outlets for local streams provided through this great dike were inadequate for the discharge of the Huai floods of 250 000 sec-ft., or more. Thus, whenever a heavy flood came down the Huai River, more or less of 10 000 sq. miles of delta farms became submerged, while the water slowly drained off through the small seaward

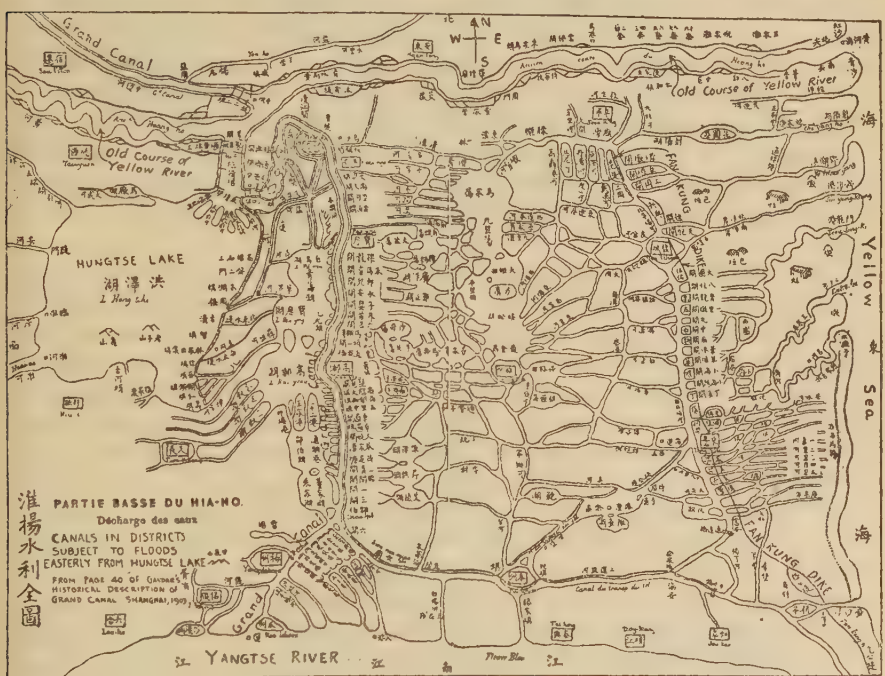

Grand Canal
Old Course of Yellow River
Old Course of Yellow River
HUNGTSE LAKE
PARTIE BASSE DU HIA-HO.
Décharge des eaux
CANALS IN DISTRICTS SUBJECT TO FLOODS EASTERLY FROM HUNGTSE LAKE
FROM PAGE 40 OF GARNER'S HISTORICAL DESCRIPTION OF GRAND CANAL SHANGHAI, 1908
Grand Canal
YANGTSE RIVER
Yellow Sea
淮揚水利全圖

FIG. 21.—CHINESE MAP OF CANALS IN HUAI FLOODED DISTRICT.

channels and down the ancient channel of the Grand Canal into the Yang-tze River. A large part of the water did not drain off, but became impounded in vast shallow lakes aggregating in area about 1 575 sq. miles.

In addition to the great Maritime Dike and the Grand Canal dikes on the east, and the dikes of the new Yellow River channel on the north, the great Ming Dike had been built south of the Huai channel about 300 years ago,

for the purpose of turning the Huai floods toward their ancient outlet. Two old Chinese maps of the region are reproduced in Figs. 3 and 21, respectively, the first showing the district flooded and the second some of the 3 200 miles of canals and watercourses, which are at various levels and serve the three purposes of transportation, drainage, and irrigation, but which fail to provide sufficient sluice-ways and main outlets to give escape of floods to the sea.

An excellent history of the Grand Canal in Kiang-su Province, with a most interesting story of the successive disasters, was published by a French priest, Father Gandar, at Sicawei, near Shanghai, in 1903. This history was derived from ancient Chinese sources and begins with a terrible deluge 2 357 years before the Christian era. Father Gandar quotes statements from Confucius about the earliest section of the Grand Canal, which was built in this district 486 B. C. For the past 500 years, he tells a story of prolonged suffering from floods, and of Chinese courage, industry, and perseverance in fighting back these floods, which has no parallel in history. Coming down to recent times, he states that within the 50 years of the Kun Yan Period, from 1746 to 1796, there were 16 years of flood, and in the 37 years, from 1844 to 1881, he tabulates the height of thirteen bad floods. After the Yellow River again migrated northward, in 1851, the floods became more rare, but the river, during its 530 years' usurpation of the old Huai Channel, had raised its bed and also had been confined by high dikes, so that, although this ancient channel was now empty, the Huai River could not now climb back into it or use it.

The speaker was invited to study the flood problems of this region, but was unable to prolong his stay in China for that purpose, and purely as a good-will offering to the friendly Chinese, he has since spent much time studying the data collected by their engineers, and some months ago ventured to submit for consideration an entirely new plan of relief, which should take advantage of the discoveries about the erosion of delta silt by the Yellow River, previously described, and which also should give to the Huai floods the shortest and straightest course possible to the sea.

The speaker also suggested that if test pits and borings should prove that the soil in this part of the delta was as easily eroded as that along the present course of the Yellow River, the erosive power of the flood itself might be trained to do a large part of the work of excavation and thus reduce the cost of the great flood channel needed to a sum within the possibilities of financing.

Under present conditions of finance in the world in general and in China in particular, it seemed hopeless to raise the $90 000 000 Mex. required to carry out the project of relief developed by the Chinese engineers. Moreover, the speaker was led to believe, through a study of all the data that he had been able to obtain, that by means of a different plan a vastly greater area of shallow lake beds could be reclaimed for agriculture than had been proposed in either the Red Cross report or the later Chang Chien report of 1919.

Tentative estimates on rather bold and broad lines, from limited data, indicated that the value of the land that could be reclaimed would become

MAP SHOWING LOCATION OF PROPOSED NEW CHANNEL OF THE HWAI RIVER
All Contours are in Meters above Mean Sea Level
PROPOSED NEW CHANNEL OF HWAI R.
Old Yellow R. Region
Canals in this region are not shown
THE GREAT DIKE OF HUNG-TZE LAKE
NORTH DIKE OF ABANDONED CHANNEL OF YELLOW RIVER
SOUTH DIKE
GRAND CANAL
Tai-chwang
Pelao
CENTRAL R.
Yao-wan
Lomo Lake
Shu-yang
Liu-Lao-kien
Hai-chow Bay
Chu-yi Lake
Hsiao-wei R.
Sha-yang-pu-shih
Hui Shu
Kwan R.
Lien-shui
Tatung Kow
Tai-kuan
Yang-chwang
Antung
Hwai-yin
Hwai R.
Hung-tze Lake
Hwai-an
Pei-ma Lake
Pao-ying
Pau-yin Lake
Hsia-yang Lake
Cha-shu Lake
Kao-yu Lake
Kao-yu
Hsia-yang R.
Hsin-yang R.
FANKUNG R.
Tou-liang R.
TIENTSIN - PUKOW R. R.
Pukow
Yang-chow
Hsiennu Miao
Kwa-chow
Sin-kiang-yin
Nanking
Chia-kiang
Tanyang
Kintan
Chang-chow
Tung-chow
Chang Tan Lake
Kao Lake
Liyang
Wu-hsih
Ihing
Tai Lake
Soo-chow
Kiating
Tai-tsang
Kun-shan
Woo-sung
Shanghai
Yangtze R.
TSUNG MING IS.
KIANGSU
CHEKIANG
ANHWEI
SHANTUNG
YELLOW SEA
DIKE

FIG. 22.

sufficient to reimburse the entire cost of the necessary drainage channel and all the other flood-protection works required in this large district.

The method worked out by the speaker is shown in Figs. 22 and 23, which present, respectively, a map of the district showing the location of the proposed new channel and a cross-section illustrating a possible method of constructing the new flood channel. This new straight channel would serve both navigation and irrigation and would also carry the discharge of the Huai River under normal conditions.

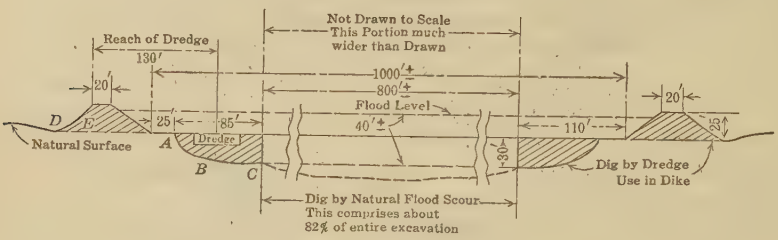
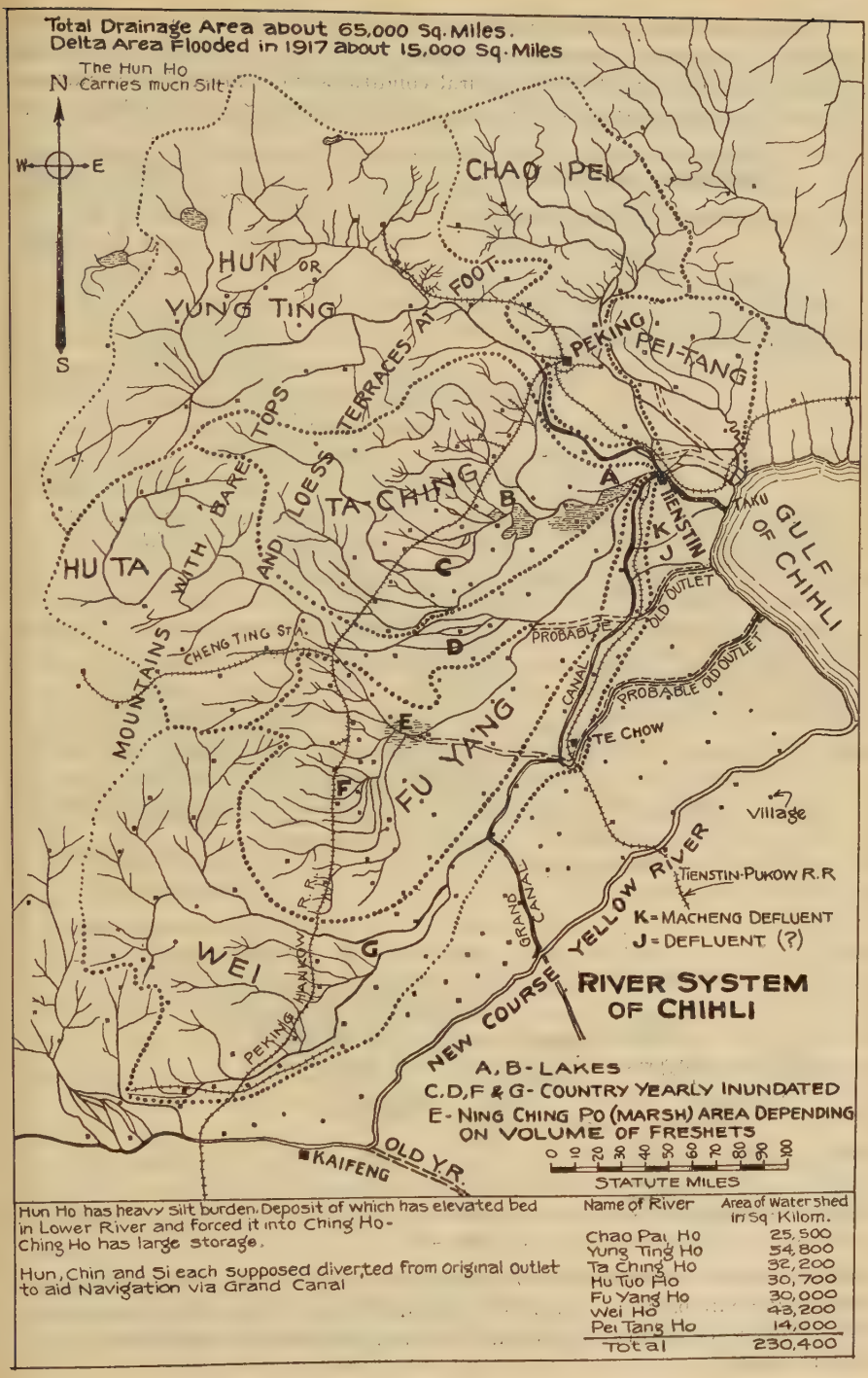


FIG. 23.

It was suggested that if borings and test pits showed favorable ground, all that need be excavated was one, or preferably two, deep, narrow channels inside the edges of the future main channel, which preliminary channels, providing barely sufficient earth with which to build the new dikes, should be cut so deep (about 30 ft.), that under the existing slope and brief yearly conditions a scouring velocity of from 5 to 8 ft. per sec. would be produced, which velocity it was believed would rapidly cut away and carry off, as long as the flood flow lasted, the silty material between these two preliminary ditches. The floods of several years were expected to complete the work gradually, aided here and there, and from time to time, by special training dikes and groynes and by men and baskets or by dragline excavation where necessary. The hydraulic estimates were worked out with considerable detail. The determination of the shape and depth of the preliminary channels for the production of maximum scour is a somewhat intricate problem of itself, but this paper is not the place for the presentation of those special problems, and this whole Kiang-su problem requires more study both in the field and in the laboratory.

Such a project would not be definitely recommended by the speaker, or its precise location laid down, until after much preliminary investigation, but the hope of accomplishing the much-needed relief to the district and of recovering 1 000 sq. miles of the most fertile land in the world for agriculture, which now lies submerged under shallow lakes, in a region where every acre is badly needed, which reclaimed land might ultimately repay the entire cost, seems worthy of extended consideration in comparison with the plan proposed tentatively by the Chinese engineers calling for an expenditure of $90 000 000 Mex. The speaker has no other interest in this project than that of being helpful and the attraction of a fascinating engineering problem.



Hun Ho has heavy silt burden. Deposit of which has elevated bed in Lower River and forced it into Ching Ho - Ching Ho has large storage.

Hun, Chin and Si each supposed diverted from original outlet to aid Navigation via Grand Canal

Name of River	Area of Watershed in Sq. Kilom.
Chao Pai Ho	25,500
Yung Ting Ho	54,800
Ta Ching Ho	32,200
Hu Tuo Ho	30,700
Fu Yang Ho	30,000
Wei Ho	43,200
Pei Tang Ho	14,000
Total	230,400

FIG. 24.

The straight course is proposed to be designed with dikes protected in some localities by groynes, somewhat as shown in Fig. 20, or on other locations, as *A-G*, *A-1*, *M-N*, or wherever natural conditions most favored construction. In some localities, instead of the stone-faced spur-dike shown in Fig. 20, it

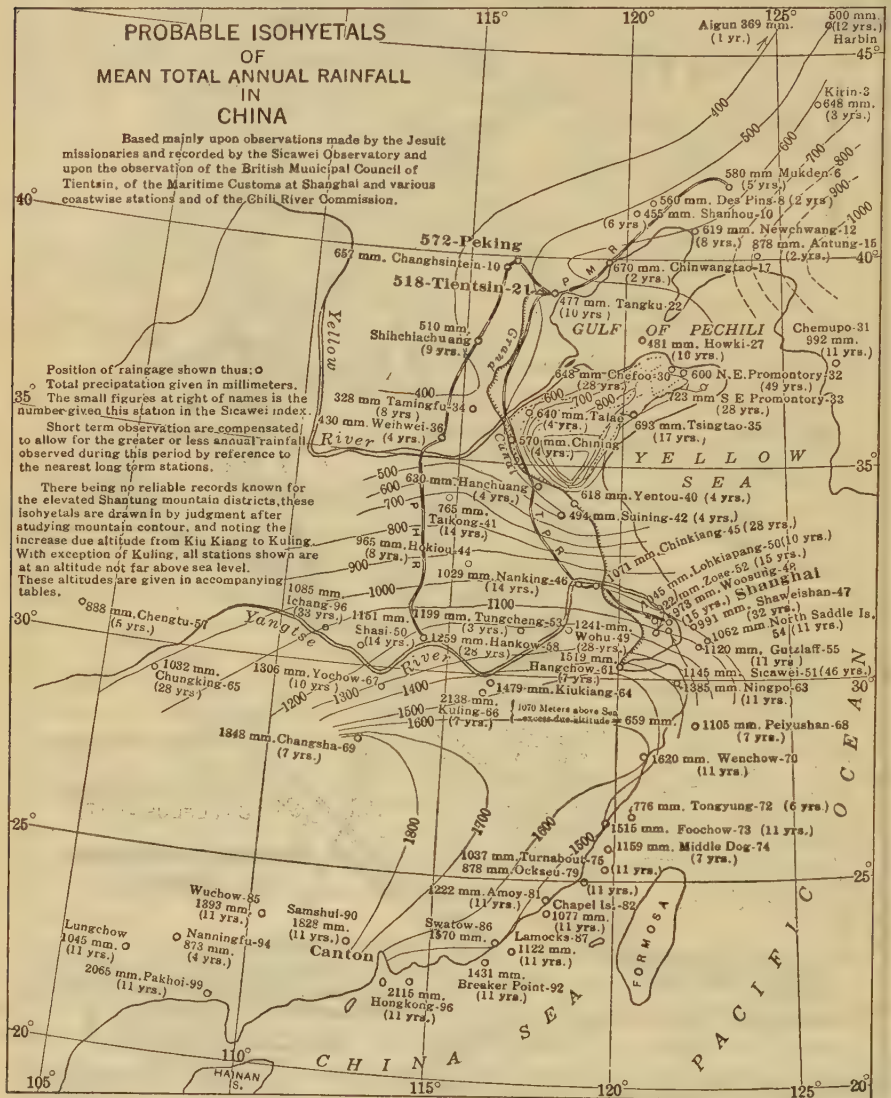

PROBABLE ISOHYETALS
OF
MEAN TOTAL ANNUAL RAINFALL
IN
CHINA
Based mainly upon observations made by the Jesuit missionaries and recorded by the Sicawei Observatory and upon the observation of the British Municipal Council of Tientsin, of the Maritime Customs at Shanghai and various coastwise stations and of the Chili River Commission.
Position of raingage shown thus: ○
Total precipatation given in millimeters.
The small figures at right of names is the number given this station in the Sicawei index.
Short term observation are compensated to allow for the greater or less annual rainfall observed during this period by reference to the nearest long term stations.
There being no reliable records known for the elevated Shantung mountain districts, these isohyetals are drawn in by judgment after studying mountain contour, and noting the increase due altitude from Kiu Kiang to Kuling. With exception of Kuling, all stations shown are at an altitude not far above sea level. These altitudes are given in accompanying tables.
572-Peking
518-Tientsin-21
Shanghai
Hankow
Canton
Hongkong
Kuling
Yellow River
Yangtse River
GULF OF PECHILI
YELLOW SEA
CHINA SEA
PACIFIC OCEAN
FORMOSA
HAINAN I.
Aigun 369 mm. (1 yr.)
500 mm. (12 yrs.) Harbin
Kirin-3 648 mm. (3 yrs.)
580 mm. Mukden-6 (5 yrs.)
560 mm. Dea Pins-8 (2 yrs.)
455 mm. Shanhou-10 (6 yrs.)
619 mm. Newchwang-12 (8 yrs.)
878 mm. Antung-15 (2 yrs.)
670 mm. Chinwangtao-17 (2 yrs.)
657 mm. Changshintein-10
477 mm. Tangku-22 (10 yrs.)
510 mm. Shihchiachuang (9 yrs.)
481 mm. Howki-27 (10 yrs.)
Chemupo-31 892 mm. (11 yrs.)
648 mm. Chefoo-30 (28 yrs.)
600 N.E. Promontory-32 (49 yrs.)
723 mm. S.E. Promontory-33 (28 yrs.)
328 mm. Taminkfu-34 (6 yrs.)
640 mm. Talao (4 yrs.)
693 mm. Tsingtao-35 (17 yrs.)
430 mm. Weihwei-36 (4 yrs.)
570 mm. Chining (4 yrs.)
630 mm. Hanchuang (4 yrs.)
618 mm. Yentou-40 (4 yrs.)
765 mm. Taikong-41 (14 yrs.)
494 mm. Suining-42 (4 yrs.)
965 mm. Hokiou-44 (8 yrs.)
1071 mm. Chinkiang-45 (28 yrs.)
1029 mm. Nanking-46 (14 yrs.)
1085 mm. Ichang-96 (33 yrs.)
888 mm. Chengtu-51 (5 yrs.)
1151 mm.
1199 mm. Tungcheng-53 (3 yrs.)
1241 mm. Wohu-49 (28 yrs.)
1259 mm. Hankow-38 (28 yrs.)
1062 mm. North Saddle Is. 54 (11 yrs.)
1120 mm. Gutzlaff-55 (11 yrs.)
1519 mm. Hangchow-61 (7 yrs.)
1145 mm. Sicawei-51 (46 yrs.)
1032 mm. Chungking-65 (28 yrs.)
1306 mm. Yochow-67 (10 yrs.)
1479 mm. Kiukiang-64
1385 mm. Ningpo-63 (11 yrs.)
2138 mm. Kuling-86 (7 yrs.)
1070 Meters above Sea = 659 mm.
1105 mm. Peiyushan-68 (7 yrs.)
1848 mm. Changsha-69 (7 yrs.)
1620 mm. Wenchow-70 (11 yrs.)
776 mm. Tongyung-72 (6 yrs.)
1515 mm. Foochow-73 (11 yrs.)
1159 mm. Middle Dog-74 (7 yrs.)
1037 mm. Turnabout-75
878 mm. Ockseu-79 (11 yrs.)
1222 mm. Amoy-81 (11 yrs.)
Chapel Is.-82 1077 mm. (11 yrs.)
Lamocks-87 1122 mm. (11 yrs.)
Swatow-86 1370 mm.
Wuchow-85 1893 mm. (11 yrs.)
Samshui-90 1828 mm. (11 yrs.)
Nanningfu-94 873 mm. (4 yrs.)
Lungchow 1045 mm. (11 yrs.)
2065 mm. Pakhoi-99 (11 yrs.)
1431 mm. Breaker Point-92 (11 yrs.)
2115 mm. Hongkong-96 (11 yrs.)

FIG. 25.

might be better to use groynes of submerged concrete sheet-piling, reinforced with large split bamboo, instead of with steel rods. It would be pleasing to the builder to obtain his reinforcement from quick-growing plantations alongside the work.

The Chihli Flood Problems

A map of this catchment area is given in Fig. 24. The terrible results of these floods of July, August, and September, 1917, have already been briefly described. Most of the delta of 12 000 sq. miles was inundated, bringing great suffering to more than 5 000 000 people and a property loss of more than $50 000 000. In general, this was said to be the worst flood in 140 years.

The conditions of soil, slope, and drainage, in this delta are substantially the same as for the deltas of the main Yellow River and that of the Huai, for all are of common origin—loess brought from the hills by floods and deposited in vast, nearly level deltas—and all have the present rivers diked in; but the flood in Chihli is of local origin, from a water-shed not distant more than about 250 miles from the convergence of the five rivers shown in Fig. 24. Therefore, the Hun River floods are quicker, briefer, and relatively more violent, considering the small water-shed, than those of the Yellow River. This Chihli water-shed, as shown by the isohyetal map, Fig. 25, lies where the annual depth of rainfall averages only about one-half that of the Huai, but, nevertheless, it can have precipitation in a single storm as much as that of the great Miami Valley flood.

From the manuscript of a preliminary report by Mr. H. Van der Veen, Consulting Hydraulic Engineer to the Chinese Ministry of the Interior, the following interesting facts are transcribed. The rainfall at Shir-kia-chuang, on the railroad at the western edge of the delta plain, near the foot of the hills, as recorded by the Administration of the Tcheng Tai Railway, showed about 40 in. of rainfall in the three months of July, August, and September. This three months' rainfall is about double the average total for the whole year in that region. Of this, about 18 in. fell in July, and on the 26th of that month, 10.4 in. fell in one day, following ten days of almost continuous rain. This was followed by 10.7 in. more in August, and nearly 9 in. in September, thus saturating the ground. Although rain-gauges throughout the drainage basin were lacking, the fact that widespread floods were simultaneous on the several converging rivers shown in Fig. 24, indicates that this one gauging presented something more than a local condition.

The best information now available indicates that, although a large part of the catchment area in the lower slopes of these Chihli hills is covered with loess earth, which has a remarkably great capacity for quick absorption of ordinary rains, the upper parts of the hills and mountains are steep and mostly bare of loess, having been almost completely deforested a few hundred years ago.

The conformation of foot-hills and plain in this drainage area is nowhere favorable to building large detention reservoirs, and no detention of water precipitated beyond the natural absorption of the soil, is practicable, except such as might be brought about by many years of reforesting. Immediate protection would seem mainly a matter of dikes and carefully planned channels and of building the farmhouses on mounds.

A partial protection by detention in ordinary floods is given by the wide distance between the existing dikes and by the extremely crooked courses of

the rivers as they meander between the dikes, all of which tends to delay the progress of the flood and to give a moderate volume of storage space when the bank-full stage is exceeded. Unequal delay on different streams tends to prevent their flood peaks from coinciding and, perhaps, it is in part this excess of channel length and excess of width between dikes in the upper part of the delta that has made the narrow, single channel within the City of Tientsin tolerable for so many years. The maximum discharge capacity of this channel at the time of the 1917 flood is estimated to have been only from 35 000 to 50 000 sec-ft., whereas the estimated discharge in the Hun River alone was upward of 200 000 sec-ft. The widespread suffering caused by this flood was almost beyond description.

The Chihli River Improvement Commission

All this led to immediate action by the Chinese Government and the appointment of the best hydraulic engineering talent available in China on an organization which developed into the Chihli River Improvement Commission. This Commission is still at work with a corps of Chinese engineers, under the advice of an English chief engineer of much experience in India, who has, for his principal assistants, two members of the Society.

The message from President Feng-Quo-Chang to the inaugural meeting of the Commission read like a verse from an ancient poem. It was brief and to the point and worthy of a land great in classic literature:

"The great flood has caused my people miseries,
The down rush of mad water has demolished embankments and dikes,
In haste let engineers be consulted with conservancy measures,
For in this way the nation as well as the government will be benefited.
The Han Emperor in person led the people to work on the damaged rivers
And in return for his zeal, Heaven blessed his reign with prosperity.
Let the Commissioner vie with the illustrious Emperor in the merits he achieved,
And secure for modern China prosperity as great."

Maps, river gaugings, and many data have been secured for the rivers of Chihli Province, particularly for those having their outlets past the great commercial city of Tientsin. As far as the speaker has learned, no definite locations or dimensions for flood-relief channels have yet been decided on, except a short cut across a bend through Tientsin, already completed, and some work in extending and deepening the defluent channel of the Ma-cheng Canal.

Several tentative suggestions have been made from time to time, and some surveying has been done for several defluent channels, by which flood water could be deflected from the Wei and other rivers down ancient shallow drainage courses, across the delta to the sea, some of which may mark the site of prehistoric channels of the Yellow River, and are shown in Fig. 5. There seemed to be a popular demand for such flood-diversion channels, so strong that it was difficult for the Commission to resist it.

The speaker inspected the site of the offtakes for some of these proposed defluents, which have been utilized to some extent in the past, but now are

clogged with silt, and was led to believe that this method of relief by defluent channels as first proposed would not be permanently successful, because of certain fundamental principles of hydraulics, which lessen the power of a stream when subdivided to cut its way, or to carry its burden of silt, as efficiently as when consolidated in a single channel. In fact, such subdivision is reversing the process of Nature, which unites streams as they proceed. Incidentally, the speaker may remark that he saw an illustration of what is likely to happen, down stream from any defluent from a silt-laden river, in the present conditions below the Ma-cheng Defluent, on the Wei River, where the defluent channel, or so-called "canal", had become so filled with silt as to be useless for navigation during the dry season, and where the main river, below the point of abstraction of water, immediately becomes narrowed or shoaled by dropping a part of its burden of silt. The vast burden of loess silt brought down by the floods greatly complicates the problem.

To a statesman or to a business man who has not carefully studied these matters of flood and silt transportation in the light of hydraulic science, it might seem that the obvious way of preventing inundations at the populous and important city of Tientsin would be to provide a channel for diverting a large part of the water before it reached Tientsin and retain meanwhile the present channels into and out from the city to the sea, but the great and uncommon burden of silt in time of flood has to be reckoned with. The speaker is inclined to believe that the best permanent relief would be found by concentrating the entire flow into the straightest and narrowest channel computed to be able to carry the maximum flood volume.

The best location for this channel is another question. Perhaps this single river channel could be safely made narrow and diked to dig itself deep and flow through the midst of this city, or it could be deflected to one side and a loop canal built from this flood channel through the city for the needs of navigation.

It is no small problem to determine how best to improve or to preserve during flood and drought the much needed, deep ship channel from Tientsin to the sea. It has been said that 2 000 years or more ago, Tientsin stood near the edge of the sea, but, because of the delta growth, it is now twenty miles distant from the sea, with a bad bar at the river's outlet. The tendency of salt water when mixed with fresh water to cause the latter to precipitate its burden of silt seems to be in evidence at the mouth of nearly all these great Chinese rivers.

Flood Waves

This type of flood problem appears to be presented in the slopes of the Shantung Mountains leading out on to the delta plain. In the course of the Grand Canal surveys, evidence was found of almost incredibly large flood volumes flowing for brief periods from relatively small catchment areas, for example, on the Wen River, which flows from the northwestern denuded slopes of the Shantung Mountains. Careful explorations were made and levels were run over a long series of flood marks along the river, which are undoubtedly authentic. Their height and slope when combined with measurements of the cross-section of the river showed, by computations from the

Kutter formula, a flood discharge at the maximum of about 330 000 cu. ft. per sec. from about 3 200 sq. miles, which is larger than the greatest flow measured at the peak of the greatest flood in the neighboring Yellow River from about one hundred times this Wen River drainage area, during the past ten years, but this Yellow River flood continues for about two months, while that of the Wen, the mountain torrent flood, continues great for only a few hours.

In 1914, the Red Cross engineers happened to have an opportunity to gauge a flood flow in the Yi River of probably 140 000 sec-ft., from an area of about 4 750 sq. miles, and there are other evidences of extremely violent brief floods from the westerly slopes of the Shantung hills.

The speaker is inclined to explain the extreme height of these floods, as well as their brief duration, by conditions not unlike those found in some parts of the Western United States which cause an extremely sudden flood, commonly attributed to a "cloudburst". In these cases, an almost instantaneous rise of perhaps 10 or 15 ft. occurs in the stream surface within as many minutes, often with disastrous effects to cattle or teams that happen to be within its reach.

A possible explanation is that under the well-known hydraulic law by which the velocity of flow in a channel increases with the depth, so that four times the depth gives double the velocity, according to its one-half power of the depth, as given by different authorities, the water in the rear, having advantage of the increased depth, travels faster and overtakes and piles on top of the earlier run-off, thus creating the great height of the advance wave. Recent authorities made the velocity increase in a higher ratio to the depth, some as the two-thirds power, some as the three-fourths power, instead of as by the time-honored Chezy formula.

This particular flood phenomenon is well worthy of more observation and of more careful scientific study than it has yet received.

FLOOD CONTROL OF THE MISSISSIPPI RIVER

By J. A. Ockerson,* Past-President, Am. Soc. C. E.

In view of the fact that, in recent years, devastating floods on relatively small streams present intricate problems in local flood control, the work of controlling the floods of the Lower Mississippi River looms as a gigantic undertaking. The Mississippi drainage basin covers an area of 1 240 050 sq. miles. The eastern extremity extends into Western New York about south of Buffalo, the western extremity, into Montana, west of Butte, the northern, into Western Canada, 70 miles north of the boundary line, and the southern extremity, at the Gulf of Mexico. Besides all or parts of thirty States, this drainage basin includes 20 000 sq. miles in Canada. It covers 1 822 miles in longitude and 1 449 miles in latitude, or 41% of the United States proper.

This great basin is divided into several tributary basins, as shown in Table 3, in which is also tabulated the mean annual rainfall in each, for a period of 25 years, as published in *Bulletin E* of the U. S. Weather Bureau for 1897.

TABLE 3.

Name of basin.	Square miles.	Percentage of whole.	Average annual rainfall, in inches.
Ohio	201 700	16	44.2
Upper Mississippi	165 900	13	31.9
Missouri	527 150	43	19.4
Arkansas	186 300	15	29.6
Red	90 000	7	39.1
Central	69 000	6	51.4
Total	1 240 000	100	29.8

It was inevitable that the gathering of water from this vast area would produce tremendous floods, particularly from the mouth of the Ohio down to the Gulf, and the people, who were attempting to reclaim the fertile basins, were unable to cope with them, although they taxed their resources to the limit. This water poured down on the 29 790 sq. miles of the delta basins of the Lower Mississippi and appeals for Federal aid were long ignored, on the theory that the protection of private property was not a function of the Federal Government.

It remained for an ex-slave, in an appeal to Congress, to call attention to the fact that the people only wanted protection from the floods forced on them from other States. "Let them take care of their own water and we will take care of ours," said he. This caused a change of view and, in time, it came to be considered as reasonable for the country as a whole to render aid. Aside from that, the reclamation of this great area of the most fertile

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part of the United States, making it habitable both as to health and exceptional crop production, was and is a good investment.

The people in the delta basin below the Ohio have expended $111 679 618 in the construction of levees since 1882, and the Federal Government has expended $47 583 368 up to December 31st, 1920. Large sums were spent by States and local interests prior to 1882, but the amount has never been satisfactorily determined. Under the present Flood Control Act, the local levee districts are required to contribute at least one-half as much as the Federal Government allots to levee construction. From 1882, when Federal allotments began, to the passage of the Flood Control Act in 1917, the local districts devoted twice as much, or even more, to levee construction than the Federal Government, and even to-day the States and levee boards spend annually, on their own initiative, much more than the contributions required by law.

The Mississippi River Commission was created by Act of Congress in 1879 and was charged with developing and executing plans for the improvement of the Mississippi River, to "correct, permanently locate, and deepen the channel and protect the banks of the Mississippi River, improve and give safety and ease to the navigation thereof; prevent destructive floods; promote and facilitate commerce, trade, and the postal service." It is a most comprehensive law and reflects great credit on the vision of its framers.

Immediately after its organization, the Commission began the study of the physics of the problems assigned to it, and exhaustive surveys, topographical, hydrographical, and hydrometrical, were undertaken. River slopes, velocities, and discharges at various stages and localities, were measured; the character of the bed and the banks was determined; the causes of bank erosion and bar building were studied; the effects of waste weirs or outlets were analyzed; daily records of river stages at many points were established; sediment observations were made; in fact, the whole range of river physics was fully covered.

While this was under way, the amelioration of conditions in the delta region from near the mouth of the Ohio to the Gulf of Mexico received immediate attention, owing to the large area subject to overflow (sometimes 60 miles or even more in width), which required protection from floods. The principal source of these floods is in the Ohio Basin, with its heavy spring rainfall and the steep slopes of its tributaries. The sources of the floods that menace the Lower Mississippi are not always in the same drainage basin, nor in the head-waters thereof. The floods of 1844 and 1903 were from the Lower and Middle Missouri and the Upper Mississippi Basins; the floods of 1912-13 were from the Lower Ohio Basin Districts. The floods of 1882-83-84 were largely from the upper parts of the Ohio Basin and its tributaries. This shifting of the locus of the floods over such great areas prevents reservoir control. The delta area lying below the flood level is shown in Fig. 26.

The advocates of flood control for this district were divided into three groups, namely, those who favored reservoirs; those who favored outlets as the only solution; and those who regarded the experience of about two

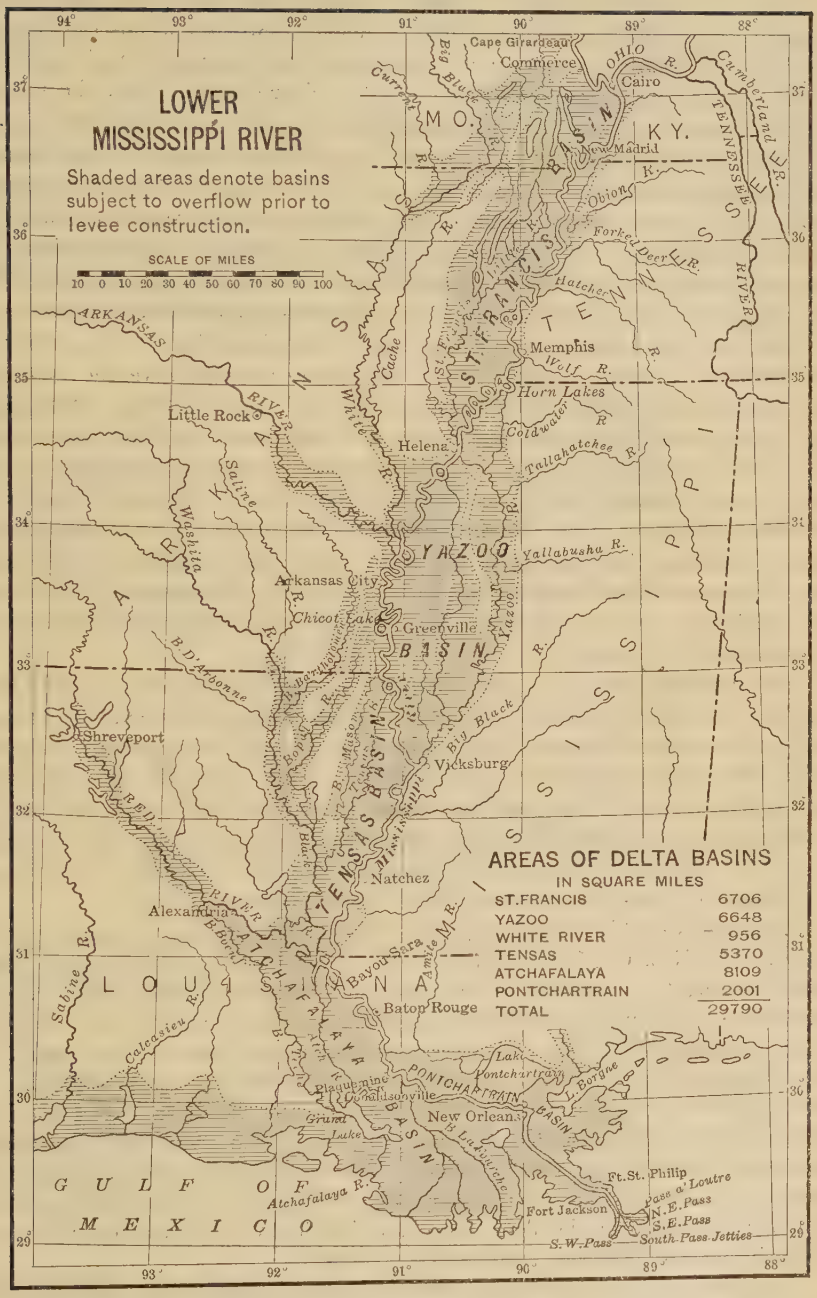

LOWER
MISSISSIPPI RIVER
Shaded areas denote basins subject to overflow prior to levee construction.
SCALE OF MILES
10 0 10 20 30 40 50 60 70 80 90 100
AREAS OF DELTA BASINS
IN SQUARE MILES
ST. FRANCIS 6706
YAZOO 6648
WHITE RIVER 956
TENSAS 5370
ATCHAFALAYA 8109
PONTCHARTRAIN 2001
TOTAL 29790
Cape Girardeau
Commerce
Cairo
New Madrid
Obion
Memphis
Horn Lakes
Helena
Little Rock
Arkansas City
Greenville
Vicksburg
Shreveport
Natchez
Alexandria
Baton Rouge
New Orleans
Lake Pontchartrain
Ft. St. Philip
Fort Jackson
Pass a' Loutre
N. E. Pass
S. E. Pass
South Pass Jetties
S. W. Pass
MO.
KY.
MISSISSIPPI
ARKANSAS
LOUISIANA
GULF OF MEXICO
YAZOO BASIN
TENSAS BASIN
ATCHAFALAYA BASIN
PONTCHARTRAIN BASIN
OHIO R.
TENNESSEE R.
Cumberland R.
Big Black R.
Yallabusha R.
Tallahatchee R.
Coldwater R.
Wolf R.
Hatchie R.
Forked Deer R.
White R.
Cache R.
Saline R.
Washita R.
Red R.
Bayou Sara
Little M. R.
Bayou Lafourche
Plaquemine
Grand Lake
Atchafalaya R.
Calcasieu R.
Sabine R.
L. Borgne
94° 93° 92° 91° 90° 89° 88°
37° 36° 35° 34° 33° 32° 31° 30° 29°

FIG. 26.

centuries with levees for partial flood control as sufficient to justify the use of them for complete control. Others asserted that the floods could not be controlled and that no attempt should be made to curb them, but that mounds should be built for occupation by the people and their live stock while the lands were flooded. The digging of another river channel from the Ohio to the Gulf equal to that of the Mississippi was also urged on Congress.

The Commission undertook a study of all these plans for flood control and concluded that reservoirs to prevent floods along the Lower Mississippi were impracticable, owing to the great number and size required and to the lack of suitable sites for such large volumes of water. Judging from the cost of the flood control of the comparatively small Miami Basin, of $32 000 000, the conclusion of the Commission as to reservoirs is manifestly correct.

The effect of outlets was given careful consideration in the study of the action of existing outlets. There can be no outlets above the mouth of Red River, that would relieve the river of flood water without returning it again at a lower point, therefore, the major portion of the delta basin would be overflowed before an escape could be reached. Extended observations and many measurements also showed that the reduction of volume, by means of secondary channels, of an alluvial stream, flowing in a bed of its own formation, will reduce the capacity of the main stream and, ultimately, will result in an increased flood height at and near the point of diversion. Therefore, the conclusion was reached that ultimately all outlets above the head of the Passes should be closed.

The depleted section of the channel just below the point of diversion can be restored to its normal capacity only by closing the secondary channel and by a quantity of water in the main channel equal to or greater than that before the diversion occurred. No scour by a lesser quantity can completely restore the channel to its original capacity, and the result will be that the permanent reduction of flood height hoped for, will not be realized. In brief, it may be said that a sedimentary stream flowing in a bed of its own formation tends to adjust its cross-section to the volume it carries.

Many cases could be cited to justify this conclusion, but a single one will suffice. The Nita Crevasse (that is, break in the levee) about 60 miles above New Orleans, La., occurred in the flood of 1890 and reached a discharge of 402 556 sec-ft., about one-third of the discharge of the entire river, but the flood stage at College Point, La., about 5 miles below, was lowered less than 2 ft. The Niagara River with its cataracts and rapids has a normal discharge of about 200 000 sec-ft., or one-half the discharge of the Nita Crevasse. The Colorado River, which created such havoc in Imperial Valley, California, has a maximum discharge of 150 000 sec-ft. These instances could be multiplied many times, all showing that disappointment and failure must follow attempts to lower permanently the flood level and incidentally the grade line of the levees by means of outlets, spillways, or weirs.

The river, at bank-full stage, will carry about one-half its maximum discharge, which is about 2 000 000 sec-ft. at the upper end of the delta. This quantity is largely dissipated as it moves down stream, filling the adjacent

unleveed basins between the levees and the highlands, at the mouths of the tributaries, and the areas between the two lines of levees.

This huge reservoir, lying between the mouth of the Ohio and the Gulf of Mexico, a distance of 1 063 miles, covers an area of 5 000 sq. miles, or about one-half the area of Lake Erie. It is a reservoir that conforms to the high-water slope of the river, in which the water is continually flowing and at flood time emptying more than 1 000 000 sec-ft. into the Gulf of Mexico. Below the Ohio, the area between the high-water banks of the river is 1 000 sq. miles, no mean reservoir in itself, particularly when supplemented by levees 18 ft. or more in height. The discharge grows less as the crest of the flood progresses down stream, owing to the quantity taken up by the reservoir. This will be particularly marked with floods of short duration.

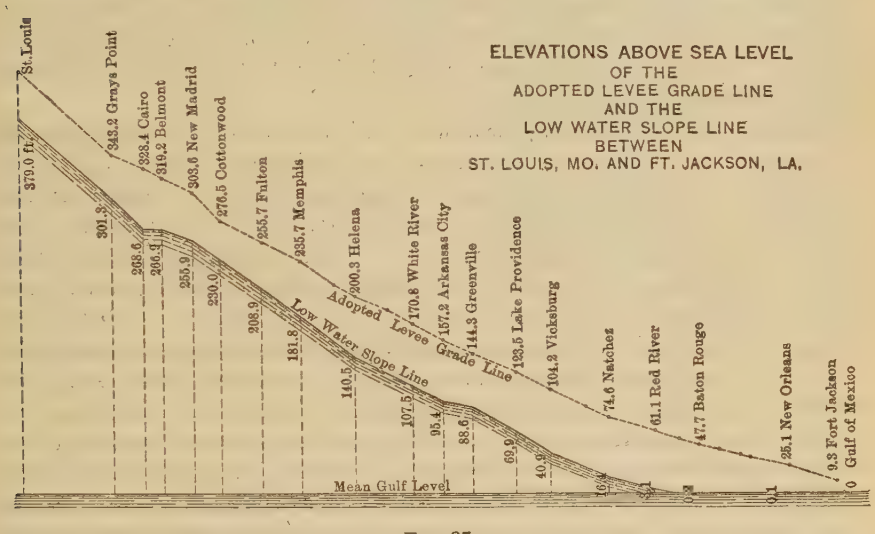


FIG. 27.

The oscillation of stage between extreme high and low water varies with the locality, as follows:

	Distance from Cairo, in miles.	Feet.
Cairo	...	55.7
Cottonwood Point	124	45.6
Memphis	227	49.2
Helena	307	58.2
Arkansas City	437	60.0
Lake Providence	542	54.1
Vicksburg	602	60.3
Natchez	706	54.3
Red River	773	53.8
Baton Rouge	841	44.2
New Orleans	965	22.6

On Fig. 27 are shown the low-water slopes of the river below the mouth of the Ohio, the distance between the successive stations being mid-river distances. The adopted provisional levee grade line is also shown. The levees fronting the great basins are called the controlling lines, and it is proposed to complete these before taking up the protection of smaller basins, most of which are hampered by difficult drainage if levees are built as a protection from floods.

The construction of levees for flood control began at New Orleans, in 1717, and by 1828 they had been extended, on the left bank, to Baton Rouge and, on the right bank, to the mouth of the Red River. Some measure of flood control has been in effect in this reach for about 200 years, but the levees in the beginning were small and breaks were frequent.

As the river banks became settled, levees were built, and, by 1860, all the great delta basins had some measure of flood protection in detached lengths of levee. At that time, the levees averaged about 4 ft. in height, whereas, to-day, they average about 18 ft.

The small levees of the early days were built by slaves, with wheel-barrows and scrapers, as evidenced by an Act of the Louisiana Legislature in 1860 which recites: "Providing for the sale, for cash to the highest bidder, by the Governor, of all slaves belonging to the Internal Improvement Department of the State, except four Engineers * * * to be selected by the Chief Engineer of the State." The word "Engineers" is capitalized in the text and relates to stationary or steam engineers, but, in itself, suggests no distinction between the technical, professional engineer and the non-technical mechanic.

In 1850, the Federal Government granted to the several States along the Mississippi all the unsold swamp and overflowed lands below the Ohio, the purpose being to provide a fund for reclaiming the inundated land, and this gave a decided impetus to the construction of levees. Even up to 1882, when the Federal Government again came to their relief, through the Mississippi River Commission, flood control by the use of levees did not meet with general approval, owing to the many disastrous failures (712 breaks in the floods of 1881-82-83), although competent Engineer Boards had repeatedly declared that flood control could only be realized by the use of adequate levees.

It was claimed that the construction of levees caused the bed of the river to be raised and, consequently, they would defeat their own purpose. A distinguished man, formerly a U. S. Senator, wrote that he was "reliably informed that the bed of the river was higher than the adjacent land," all caused by the construction of levees. To secure definite information on this subject, extended investigations were carried on, that resulted in establishing the fact that there has been no progressive elevation of the bed of the river due to levee construction. On the contrary, the latest comparison of cross-sections shows that there is a well-defined tendency toward a cross-section of greater capacity, including a lowering of the bed of the river.

To insure flood control on the Lower Mississippi River, it became apparent that levees, earthen dikes, or embankments, must be used, and this plan was adopted by the Commission, the purpose being to protect the delta basins

from any combination of tributary floods such as have occurred within historic times.

After centuries of successful use of levees for flood control on the Po and Theiss Rivers in Italy, the Nile in Egypt, and even in Louisiana, it was hard to account for the organized opposition to levees along the Lower Mississippi. This opposition, however, has largely disappeared with the success of the flood protection already obtained from the partly completed levee system.

Prior to the construction of levees, the floods of the river covered a width of 60 miles, as shown by the shaded areas of Fig. 26. The floods, laden with sediment, overflowed the channel and deposited the major portion of their load near the river banks, and this, in time, developed a land slope, away from the river, of from 3 to 15 ft. per mile. The natural drainage, therefore, is inland, toward the highlands bounding the delta. The drainage there enters tributary streams along the foot-hills, and these, in turn, empty into the Mississippi at the lower end of the several basins, such as the St. Francis, the Arkansas, the Yazoo, etc. Nature has greatly simplified the problem of flood control by making the drainage of the flooded lands, in the larger basins, comparatively simple.

To bring the vast expanse of water covering the great basins under control between levees, sometimes less than a mile apart, and further complicated by tributary inflow, made the determination of the required levee grades a difficult problem, but the elevations deduced, proved to be fairly close approximations. As 1 500 miles or more of levee are required to protect the great basins from floods, it was early decided to give as great a measure of protection each year as the funds available would permit, rather than attempt to begin at one end and complete the levee as the work progressed.

At present, the levees are completed to standard grade and section for about 500 miles. The entire length of the controlling levee line is to a grade that will withstand normal floods and the practice is to complete the levee to standard grade and section, from the up-stream limits of the respective basins, as the work progresses.

The standard levee section adopted has a crown width of 8 ft. at 3 ft. above the highest flood stage and side slopes of 1 on 3, supplemented on the land side, by banquettes 20 ft. wide for levees from 10 to 13 ft. high, 30 ft. wide for levees from 13 to 16 ft. high, and 40 ft. wide for levees more than 16 ft. high, the tops of the banquettes being from 5 to 8 ft. below the tops of the levees.

The ground on which the levee is built is carefully prepared, first, by the removal of trees, brush, logs, and débris of all kinds. It is then grubbed to insure the removal of stumps, roots, drift, logs, and other unstable matter, so that the foundation will be of clean, compact, homogeneous earth. A muck ditch, about 5 ft. wide and 6 ft. deep, is then cut along the axis of the levee as a further insurance of the elimination of perishable material. The surface is then broken and turned to a depth of 6 in. On this foundation, the levee is started full width and carried regularly, in layers not exceeding 3 ft. in thickness, to the full height. Only clean earth, free from foreign matter, is used in the levee.

The shrinkage allowance is 15% of the net fill, for scraper work, 20% for wagon work, and 25% for dragline and other machine scrapers. This allowance for shrinkage is disposed on the top and the slopes so as to leave plane surfaces from the edge of the crown to the base, when shrinkage has ceased.

The earth is taken from a borrow-pit, usually on the river side of the levee, leaving a berm 40 ft. in width. The pit, beginning at the berm, is 3 ft. deep and from that point outward may be deepened on a slope of 1 on 50 and excavated so as to afford thorough drainage at the back of the pit.

After completion, the levees are sodded with Bermuda grass which affords excellent protection from erosion by rain. Where levees are exposed to wave action, a layer of concrete about 4 in. thick or a board protection is used to break the force of the waves. Severe tropical Gulf storms below New Orleans make necessary substantial protection of the levees from wave action.

The bearing power of the soil on which the present high levees are built is, in some cases, not equal to the load placed on it and subsidence, which is not easy to overcome, takes place. Subsidence sometimes results in the bulging up of the adjacent soil several feet in height and an embankment, to hold a head of perhaps 20 ft. of water on a foundation of that kind, is far from satisfactory. Subsidence of that character in time of flood would be disastrous, but, fortunately, it occurs during the construction period when precautions are taken to overcome it by adding more material.

The extent of saturation in a levee depends on the character of the soil of which it is built, the length of time it is exposed to the flood, and the depth of water against it.

Experiments show that the hydraulic gradient, or the line between the wet and the dry earth in a levee, has a slope of about 1 on 5. To make the levee secure, this line should fall well within the base, hence the necessity of reinforcing the levee with a banquette.

The wheel-barrow and scraper have been superseded by the dragline, by cableway excavators, and by dump wagons hauled by tractor; the last promises largely to replace mules and wheel scrapers. Caterpillar traction, for large dragline machines, is in use and is regarded favorably. The tractor dump wagons have a capacity of 3 cu. yd. and are filled by steam shovels or by the ordinary elevating loader, also drawn by tractor.

The revolving dragline levee machine is equipped with a 5-cu. yd. bucket and a 165-ft. boom. The cableway machines have a span of 600 ft. or more and carry a 6-cu. yd. bucket. It is estimated that, under favorable conditions, one of these large machines will handle 400 000 cu. yd. during an average working season, the length of the season depending on the amount of rainfall and the duration of high-stage river conditions.

Much of the levee work is through heavy timber which must be thoroughly cleared and grubbed so that stumps or roots will not interfere with the operation of the bucket. As the embankment must be free from such foreign matter, the closest watchfulness is required to prevent a stump, hidden in the huge bucket of earth, from being dumped into the embankment.

During prolonged floods, sand boils occur on the land side, sometimes several hundred feet from the levee. These are caused by water, flowing through

the soil underneath the levee, breaking through to the surface in the shape of bubbling springs. As long as these springs carry only clear water, they are harmless, but should sand or sediment appear, prompt steps are taken to check them by impounding the flow. Earth-filled bags are used to create a head of water on the river side sufficient to check the flow.

During excessive flood periods, low-grade levees are topped from 3 to 5 ft. with earth-filled bags, which have been effective in preventing the levees from being overtopped, an active cause of crevasses and devastating floods. The erosion of the river banks has been and still is active in the destruction of levees. Fortunately, this is far more active during falling stages of river and with few exceptions never occurs during flood stages. In many places, active bank erosion has made it necessary to rebuild the lines of levees several times; particularly is this true of the earlier work in which the levees were placed too near the river bank. The present practice is to locate the levee so that it will be safe from normal bank erosion for at least 20 years.

The remedy for bank erosion is the revetment which has been developed by the engineers of the Mississippi River Commission. There are three general types, namely, willow mattresses, articulated concrete mats, and solid concrete mats.

The willow mattress, in general use above Vicksburg, Miss., which is of the fascine type, is made up of bundles of willows held together by wire cables and heavy poles bound to the fascines by galvanized wire. These mats are generally made in lengths of about 1 000 ft. and are 250 to 350 ft. wide, depending on the depth of the water. Such a mat requires more than 100 acres of willow brush for its construction.

The head of the mattress is a bundle, about 3 ft. in diameter, of strong hardwood poles, 5 to 8 in. in diameter, with a length equal to the width of the mat, which forms the connecting link between the mooring cables and the mattress. The mooring barges are placed normal to the shore and serve to protect the mattress from accumulations of drift while it is afloat and also to support the head of the mattress while its construction is in progress. The $\frac{1}{4}$-in. steel weaving strands are also attached to the mat head.

For fascines, brush from 2 to 6 in. thick at the butt end, is placed in the forms, with the tops in the first layer in one direction and for the second layer in the opposite direction, care being taken to break joints. Enough brush is used to make a fascine 12 to 16 in. thick, when tightly compressed with a binding of No. 12 wire at intervals of about 8 ft., the length of the fascine being the same as the width of the mat. It is then moved down the ways to its proper place in the mattress, where the weaving strands are passed around it and also around the bottom longitudinal cables running the whole length of the mat.

The fascine is drawn taut to its neighbor by means of block and tackle, using a special clamp for the purpose. This process is repeated until the required length of mat has been reached. Rows of poles extending the entire length and width of the mat are then placed at intervals of 16 ft., and are securely lashed to the fascines by steel cables. The poles serve to keep the

rock from sliding off the mat while it is being sunk to the bottom of the river, and also to add strength.

The ballast required for sinking and holding the mats in place amounts to 1 600 lb. per square of 100 ft., and consists of stone blocks, each weighing about 40 lb., evenly distributed over the surface. An essential feature of these mats is that they must be woven so closely and so thick as to prevent the current from scouring through them. Fig. 28 shows a mat in the process of weaving. In sinking, the whole surface is first uniformly covered with as much stone as the mat will sustain, after which the head is weighted and lowered to the bottom and the sinking of the mat follows from the upper end down stream, by uniformly distributing stone from barges as the mat sinks.

The mat now covers the subaqueous bank from the shore line and extends well beyond the thalweg. The above water bank is then graded to a slope of 1 on 3 or 4 and paved with 10 in. of hand-laid riprap or with 4 in. of 1:3:6 concrete in lieu of the stone. The willows for the fascine mats described are cut, loaded on barges, and transported to the site of the revetment, where they are unloaded as the weaving of the mat progresses. This work requires a great deal of floating equipment.

Below Vicksburg, a type of framed mat is used, that is built on the river bank where the willows are cut and then is towed to the place where it is to be used for revetment. The mat is a brush raft made in sections, 100 by 150 ft., and when thoroughly compacted is 16 in. thick. In building this mat, frames are laid at 10-ft. intervals across temporary launching ways. These frames are filled with three layers of willows from 3 to 6 in. in diameter at the butt end, the middle layer being placed at right angles to the upper and lower layer. These layers are tightly compressed and a top frame is placed and firmly connected to the lower frame by posts and wooden pins. Neither cables nor wire are used in these framed mats, except for connecting several cribs together, in order to obtain the desired size for sinking.

The great quantity of willows needed for revetment soon revealed the fact that the supply could not long meet the demand, and studies were begun to develop other types and materials for use in bank revetment. Engineers naturally investigated the possibility of the economical use of concrete. Although many difficulties were experienced, two types were developed, one of which has been tested to such extent as to justify the belief that it will prove to be economical and stable.

The articulated concrete block system was developed by the engineers of the Vicksburg District. The blocks or mat units are 3 in. thick, 11½ in. wide, and 3 ft. 11 in. long, reinforced with 12-in. wire mesh. This type has an advantage in that the block units can be prepared in advance, when river conditions are unfavorable for placing the mats. A 1:2:3 concrete is used for the blocks, and, three hours after pouring, the forms are removed and relaid for the next pouring.

In sinking, the mats are laid in position on a mat-building platform and fastened to a ½-in. wire rope at intervals of about 4 ft., thirty-five units forming a section of mat 25 by 140 ft. This section is launched and is fol-

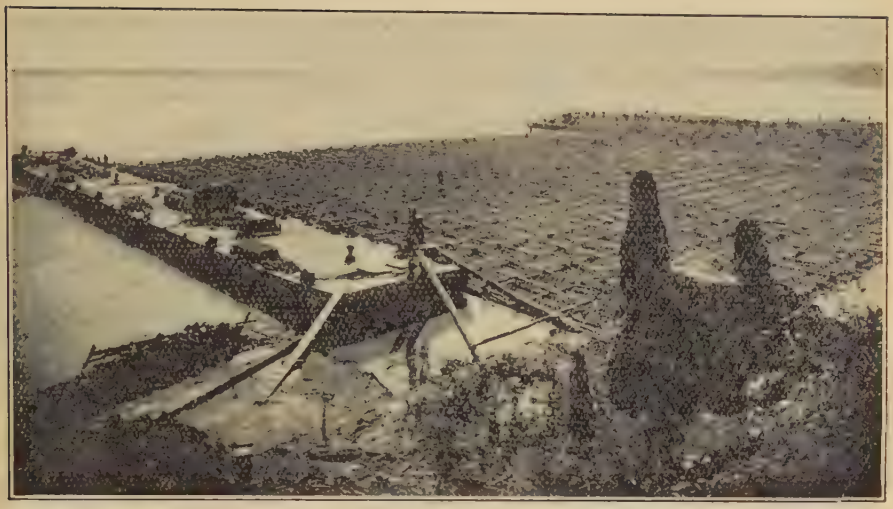


FIG. 28.—VIEW OF MAT IN PROCESS OF WEAVING.

FIG. 29.—ARTICULATED CONCRETE MAT PREPARED FOR LAUNCHING.

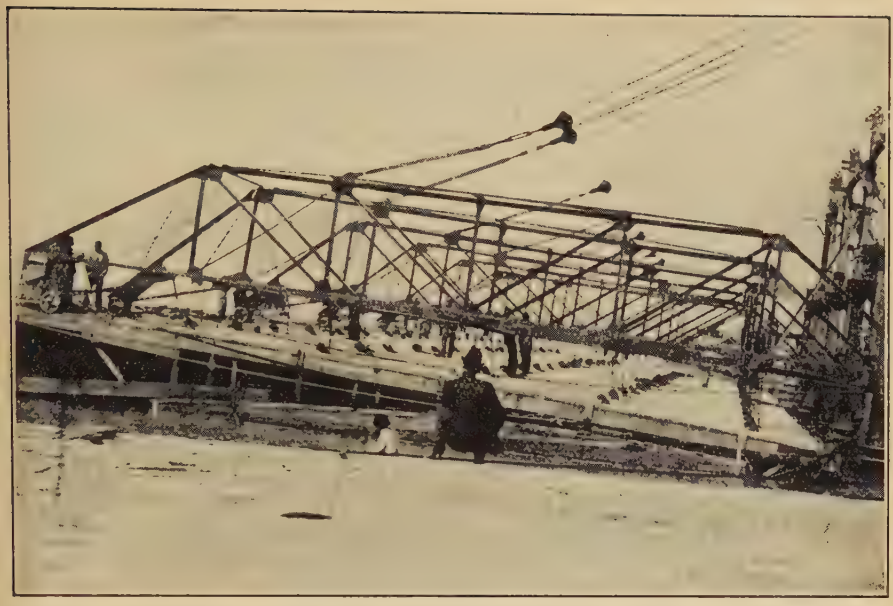
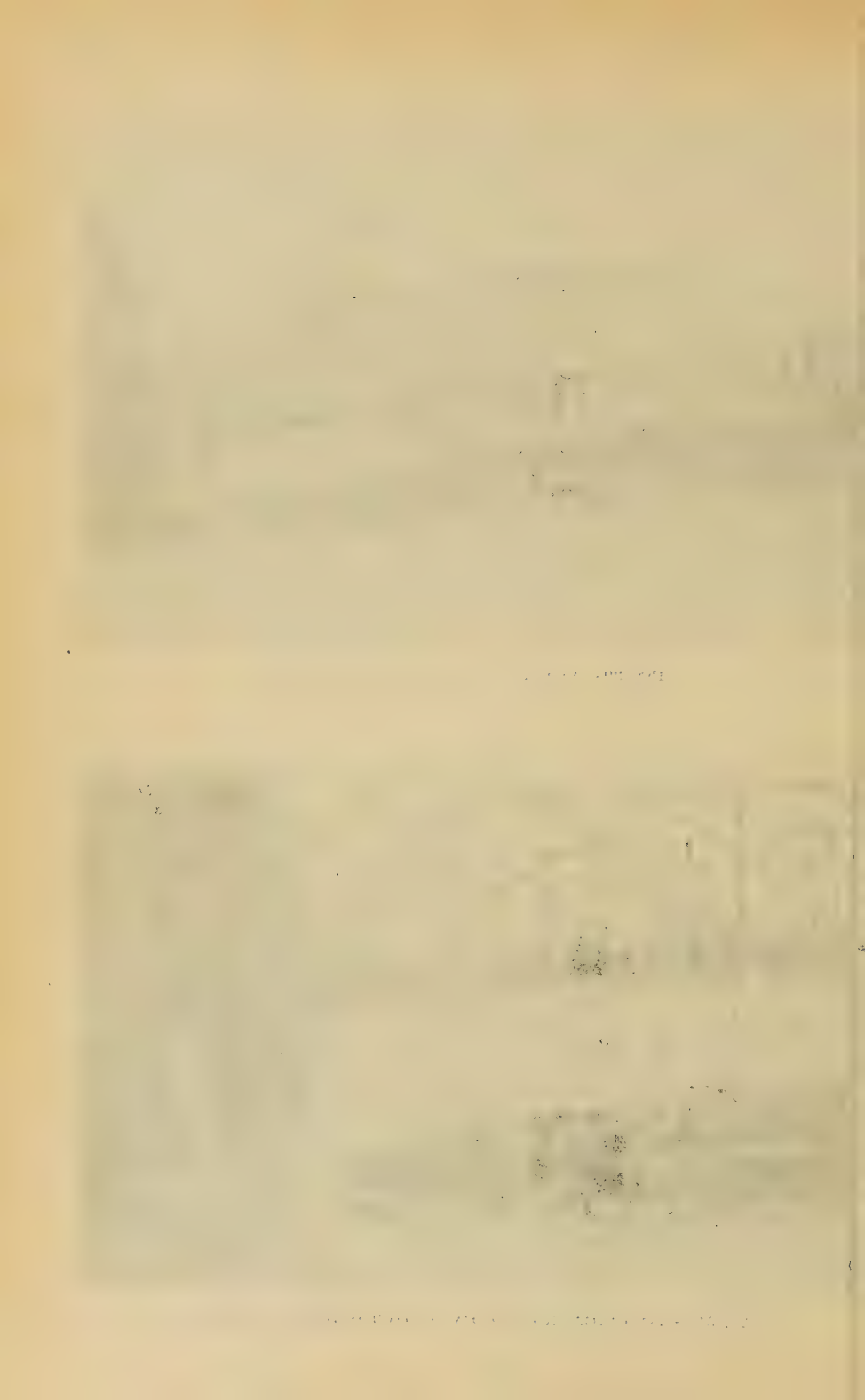


Fig. 30.—View of Mat on Launching Barge.

Fig. 31.—Mat Partly Suspended as Launch is Being Withdrawn.



lowed by succeeding sections, all of which are connected, until the required length has been obtained, the whole resting on the river bottom. An idea of the mat in process of launching may be gained from Fig. 29. This type can be laid at high water in case the upper bank has been previously graded to the proper slope. About 20 000 lin. ft. of this revetment has been successfully laid at a cost somewhat greater than the fascine willow mat.

Another type of concrete mat, developed in the Memphis District, provides for reinforced concrete laid in sheets, 50 by 150 ft. by 3 in. thick. This type of mat is sunk in a semi-plastic state from supporting trusses, by pulling the launching barge out from under it and lowering it to the bottom. The mat on the launching barge is shown in Fig. 30. The mat, partly suspended, as the barge is being withdrawn, is shown in Fig. 31. The supporting cables are tripped at the same instant the slab is in position for sinking, a short distance below the water surface. With this kind of plant, it is estimated that 4 000 lin. ft. of channel mat can be laid per month at a cost not far in excess of the willow mat.

This type of mat has not yet been tested to such extent as to develop fully its defects. When these are discovered and remedied, it seems probable that its economical use in bank revetment will result.

Methods of bank revetment have been discussed only in a general way, as this paper is primarily on flood control and relates more directly to the navigation feature of the work of the Commission, which requires it to "correct, permanently locate, and deepen the channel and protect the banks of the Mississippi River." The cost of revetment during the period of high prices incident to the World War, has been about $73 per lin. ft. of river bank. Prior to 1918, the cost was about $34 per lin. ft.

The Substantial Benefits Resulting from Flood-Control Work.—In what was formerly the area subject to overflow in the States bordering on the Lower Mississippi River, there have been extraordinary developments that must be largely credited to successful flood-control work. Without such work, even moderate development would have been impracticable.

In order to ascertain the extent of the benefits of flood control under the Mississippi River Commission, data have been taken from the United States Census reports of 1900, 1910, and 1920, that relate to the increase in farm areas, farm values, and population, as compared with like items in the non-delta parts of the States that are above the reach of Mississippi River floods.

The counties lying wholly or largely within the overflow basins of Missouri, Arkansas, Mississippi, and Louisiana, are included in these investigations. The census data are given by counties, a few of which contain considerable hill areas, and a sharp line as to flooded area alone, which is by far the major part of the area considered, cannot be drawn.

At the lower ends of the delta basins, the Upper St. Francis, Lower St. Francis, Yazoo, White, and Tensas, where tributary streams enter the Mississippi River, the levees are not completed, and the delta lands are subject to overflow to some extent from back-water, but the counties in which these lands are situated are included in the general investigation of benefits. With limitations in mind, the favorable results of the work done should be highly

gratifying. The delta counties lie in the area formerly overflowed from the Mississippi River.

The State of Missouri has 114 counties, 6 of which lie in the delta, known as the St. Francis Basin. In these six delta counties, the total value of "all farm property" in 1900 was $25 118 167 and, in 1920, it was $170 079 705, a gain in value of $144 961 538. The percentage of increase is about two and one-half times that of the State at large. The total acres of "improved land in farms" in 1900 was 604 475 and, in 1920, it was 1 072 133, a gain of 467 658 acres, which is more than one-fifth of the gain in all the other counties of the State combined.

The average price per acre of "land in farms", has increased from $19.72, in 1900, to $101.37, in 1920, whereas in the State at large the increase has been from $20.46 to $74.60. The population has increased 55 912 in the same period, whereas 89 counties show a decrease in population and more than one-fifth of the total gain for the entire State, including cities, belongs to the six delta counties.

The State of Arkansas has 65 counties, 7 of which lie along the Mississippi River, largely in the delta district, but also embrace considerable areas of hill lands. In these seven counties, the total value of "all farm property" in 1900, was $22 602 988, and, in 1920, it was $168 390 215, a gain of $145 787 227, which is about one-fifth of the increase for the entire State. The total acres of "improved land in farms" in 1900 was 592 151 acres, and, in 1920, it was 1 024 946, a gain of 432 795 acres, which is about one-fifth of the increase for the entire State. The average price per acre of "land in farms" increased from $12.32 in 1900 to $77.34 in 1920, whereas in the State at large, the increase was from $6.32 to $34.86 per acre. The population increased from 120 079 in 1900, to 220 442 in 1920, a gain of 100 363, which is about one-fourth of the total gain for the entire State. The cotton acreage increased from 341 222 acres in 1910 to 539 550 acres in 1920, with an increase of 69 493 bales of cotton in the same period.

The State of Mississippi is divided into 82 counties, 12 of which lie in the delta. In these delta counties the total value of "all farm property" in 1900 was $60 071 432, and, in 1920, it was $442 481 342, a gain of $382 409 910, which is equal to one-half the gain for the entire State. The total acres of "improved land in farms" in 1900 was 1 466 413, and, in 1920, it was 2 067 384, a gain of 600 971 acres, which is equal to one-third of the gain for the entire State. The average price per acre of "land in farms" increased from $17.29 in 1900 to $121.61 in 1920, whereas in the State at large, the increase was from $6.30 to $35.27 per acre. The population increased 107 850 from 1900 to 1920, whereas 49 of the counties show a decrease. The population of the State at large increased 293 348, including the cities, but 38% of the gain was in the delta counties. The cotton acreage increased from 1 023 353 acres in 1910 to 1 209 639 acres in 1920, or a total of 186 286 acres, with an increase of 64 026 bales of cotton in the same period.

The lower end of the Yazoo Basin is subject to overflow from back-water, as the controlling levee line is not completed for a distance of 18 miles, and the completion has been deferred so long that a considerable area of cultiva-

tion has been abandoned. Owing to this and the ravages of the boll weevil, the development is greatly reduced from that which would have been realized under normal conditions.

The State of Louisiana cannot be analyzed in the same manner as the other Lower Mississippi River States, because one-third of its area is alluvial or delta land, its years of settlement are measured in centuries, and some measure of flood control has prevailed for a like period. There are about 2 000 000 acres still subject to overflow from the Mississippi and the full benefits from flood control will not be realized until this vast area has been reclaimed.

Thus far, the benefits are largely the result of the more substantial levees, which give added security to the lives and property on land that has long been in use, rather than to any great extension of new areas of farm land.

The benefits are reflected in the farm land values, which have increased from an average of $15.05 per acre, in 1900, to $46.51 per acre in 1920, as derived from fifteen parishes fronting on the Mississippi River.

The Commission has assigned to a special committee of its members the task of preparing a full report of the work done under the Mississippi River Commission and the results attained, "in order to bring together with a view to publication and make available the great mass of valuable data relating to river hydraulics, covering physical investigations of the varying elements that make up the regimen of the Mississippi River, as well as the methods and instrumentalities that have been developed and tried out by the Mississippi River Commission and its assistants in channel improvement and flood control of the river." This work is now in progress.

Since the paper was presented, the flood of 1922 has passed, and this record would not be complete without some further account of the effects of that flood. Col. Millis* also has brought out some data relating to his experience about 30 years ago, which needs further consideration. In the lapse of so many years, great progress has been made in flood-control work, and much that was then in doubt is now in the realm of practical certainty, through constant study of river phenomena.

Col. Millis holds that the determination of the limits of future flood heights on the Mississippi River is impossible, one reason being "the constant increase of the cleared and cultivated areas within the limits of the valley and the extension of artificial drainage."

This is really insignificant when compared with the increase in flood height due to bringing the overflowed area, about 60 miles in width, into a channel between levees 1 mile or so apart. The magnitude of the flood is not now measured by comparing the present stage with that of earlier unconfined floods, but by comparing the quantities of discharge.

The Mississippi River Flood of 1844, when the contributing country was largely in its primitive state, was several feet higher than any flood since that time. Other similar instances indicate that too much stress is laid on the effect of clearing and draining in increasing flood heights in great streams like the Lower Mississippi.

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* See p. 1544.

It should be borne in mind that the available section of the river is largely increased by the levees, so that although a bankfull stage of the river carries about 1 000 000 sec-ft., the levees at standard grade carry more than double that quantity. Extended observations also show that the section of the river bed proper is gradually increasing due to confined floods.

The disaster due to the flood of 1922 has been grossly exaggerated. The flood was safely conducted for 700 miles without a break in the controlling levee line, although less than one-third of its length was up to standard grade and section, and no break occurred in such standard section. The

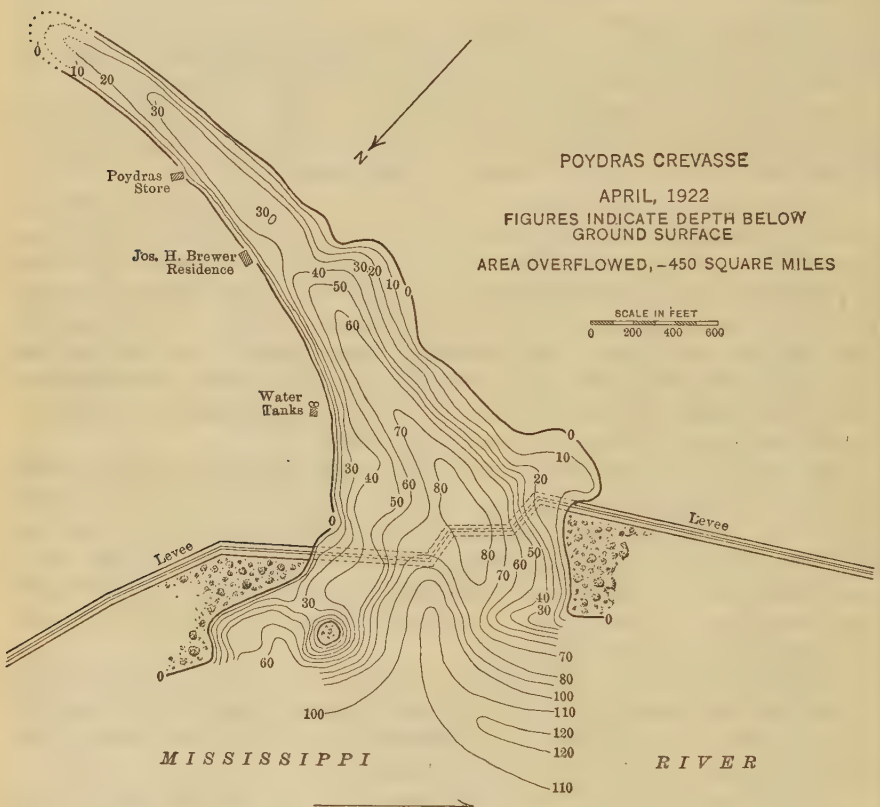

FIG. 32.

levee at Wecama, opposite Natchez, 700 miles below Cairo, Ill., where the break occurred, required 180% more yardage to bring it up to the standard section. The local flood height was reduced about 2 ft. by this break, but the escaping waters returned to the Mississippi again at the mouth of the Red River and served to increase the flood height below. This is a definite case of a spillway which was detrimental to the interests of the lower reaches of the river.

The break in the levee, which occurred 12 miles below New Orleans, at Poydras, La., on April 27th, 1922, is in a sharp bend of the river and reached

an ultimate width of 1 300 ft. and a maximum depth of more than 100 ft. near the river. The quantity of flow was not measured, but several observers estimated it to be about 400 000 sec-ft. The outflow of this large quantity covered an area of about 8 sq. miles of cultivated land and many square miles of the adjacent swamp land. The crevasse lowered the stage of the river at New Orleans about 1.6 ft. This does not offer much encouragement for the use of controlled spillways as the ultimate step in the work of flood control. The great depth of scour, shown in Fig. 32, illustrates some of the difficulties in the construction of controlling spillway work.

The axis of the cut followed along the highest part of a ridge leading from the river back toward Lake Borgne. The ground surface from this ridge southward has a slope of about 5 ft. in the first 1 500 ft., the same slope as that which exists from the river bank inland, the surface of the ground at the river bank being about 7 ft. above mean Gulf level. The distance from the river to the swamp lands averages about ½ mile.

At the time of the break, the surface of the water in the river at the site was about 20 ft. above mean Gulf level, and the short distance from tide-water in the swamps gives a tremendous slope that necessarily creates havoc in an alluvial soil.

A discharge measurement of the river made just above New Orleans on the day before the break, showed 1 300 000 sec-ft. and a mean velocity of 6.6 ft. per sec., the width of the discharge section being 2 570 ft.

A detailed survey was made of the river bed above and below the crevasse, about three months after the break occurred, but the results are not available at this time. A comparison between this survey and one over the same locality made a few months earlier will be of interest.

There were no breaks in the levees of the Commission between the Red River and a point 12 miles below New Orleans, a distance of 215 miles of effective levees, and this break was caused by the levee caving into the river and was not due to any defect in the levee itself. The flood of 1922, which exceeded all previous flood stages below the mouth of the Arkansas River, was controlled to such extent that more than 20 000 sq. miles of the delta area was protected by levees from overflow.

There is no uncertainty as to the outlay that will be necessary to make the levees safe for complete flood control, and this outlay is fully justified by the benefits accruing to the interests involved. Neither is there material uncertainty as to the grade line to which the levees must be constructed. The levees along the Po and the Rhine have been built for several hundred years and for a great many years there has been little if any change in grade line, therefore, one is justified in speaking with confidence regarding the levee grade line of the Mississippi River. The present provisional grade line has never yet been reached by the floods, and no increase in height is now contemplated, except to meet local conditions such as are due to shifting of the channel through chutes or cut-offs.

The levees above the Red River have safely carried 1 800 000 sec-ft. for hundreds of miles, and even past Natchez where the river section is only 2 300 ft. wide. Furthermore, the levees below Baton Rouge average only

about one-half the height of those in the middle river districts, and the soil available for levee construction is much better than that of the districts between Memphis and Vicksburg, so the claim that complete flood control for the Lower Mississippi River can be secured by means of levees is fully justified. Flood control of the Lower Mississippi without levees is impracticable, no matter what supplementary works may be added in the future. It is true that "efforts to effect complete flood control and security are still short of satisfactory consummation," but good progress is being made, as far as available funds will permit, and the results are reflected in the greatly increased measure of protection shown in the flood of 1922, as compared with floods of former years.

The time arrived many years ago when "serious consideration" began as to the best means of flood control, and that consideration has never lapsed, but has continued day by day and year by year through exhaustive observations, the accumulation of data, and the careful study of causes and effects.

For many years, the Mississippi River Commission has devoted much attention to questions relating to practicable flood control, among which is the utility of outlets and spillways for reducing the height of floods.

The effect of many levee breaks (crevasses) on the bed of the river and on the stage has been measured and studied, surveys have been made in detail of several possible sites for spillways on both sides of the river, above and below New Orleans, covering borings, character of initial controlling works required, embankments needed to guide the escaping floods to tidewater, and width and depth of spillway necessary to carry off sufficient water materially to affect the flood height.

With all these and many other data bearing on the question, the conclusion is reached that such devices greatly increase the flood hazard, have only a limited effect on the flood height, are expensive to construct and maintain, and would add new and serious dangers to the levees themselves. Therefore, the use of spillways for the purpose of reducing flood heights is impracticable.

Other organized investigators, under Government auspices, reached similar conclusions long before the Mississippi River Commission was created. The Commission has made far more extended investigations than any of its predecessors, and these investigations are still going on, without prejudice for or against any theory, but merely seeking facts.

Many great floods have passed New Orleans without doing material harm, but each recurring flood brings with it undue alarm, and many schemes are evolved for mitigating the looked-for disaster. The fact that no great harm has ever yet come to the city should be at least encouraging.

To convey a flood of 2 000 000 sec-ft. without damage for a distance of more than 1 000 miles through a delta plain, is admittedly a herculean task, but the measure of success even now attained, should give ample assurance that it can and will be accomplished.

It has been said that "recent floods have brought to the attention of many thinking men the great necessity for a scientific approach to the solution of the river flow and flood problems" and that "no adequate scientific study has

been made of this important problem." This is an assumption that is wholly unwarranted and widely at variance with the facts.

Able, thinking men of the highest scientific attainments have given many years of study to the flood-control problems of the Mississippi River, and an enormous volume of reliable data relating to every phase of river phenomena have been accumulated and digested. No river in the world has been studied so thoroughly, yet, in spite of all this, the press is advised "that there is a woful lack of data regarding the Mississippi River."

Those who are charged with conducting this gigantic work have the fullest confidence, and this confidence is shared by the vast majority of the people affected, that the carrying out of the plans now under way will "provide a great measure of security" and insure "against widespread loss and disaster" such as has sometimes prevailed in the past. There is a practical agreement among all those familiar with the problem, that the plans under way are correct in principle and should be carried to completion as early as practicable.

MISSOURI RIVER BANK PROTECTION AT OMAHA, NEBRASKA

By Roy N. Towl,* M. Am. Soc. C. E.

Current retards have been used with good results at Omaha, Nebr. All other types of bank protection heretofore used have been destroyed by the Missouri River, which has swift scouring flood currents, a shifting bed of sand, and alluvial banks subject to erosion.

The current retards, or pervious dikes, are made of whole trees (from 8 to 30 in. in diameter), built out from the shore a distance of 100 to 300 ft. The butts of the trees, which are placed up stream, are tied together with steel cables, held by bank cables running to "deadmen" on shore, and permanently secured with interlacing pile cables to a row of submerged concrete anchor-piles placed 35 ft. apart and 100 ft. up stream from the retard. By using these reinforced concrete anchor-piles, the problem of security against deep scour has been solved.

A pile, 20 ft. long and 14 in. square, has a center 4-in. jet pipe reduced to a 1-in. nozzle. It is equipped with three ½-in. upturned jets on each side. With a constant water supply at a pressure of 200 lb. per sq. in., the pile becomes an hydraulic giant excavator.

The center jet digs down and the twelve side jets lubricate the sinking pile and remove side friction. Within 10 or 15 min., the 20-ft. pile is sunk to a depth of from 75 to 100 ft., which is below the limits of scour of the river. The 4-in. hose connection is removed without shutting off the water supply. Six or more cables are attached to each anchor-pile before sinking.

The pile is not hammered and rests in final position in perfect condition. As soon as the water is shut off, the material through which the pipe has penetrated settles firmly around it.

A pile of any length can be sunk to any depth through any material that can be displaced under water pressure. The length and weight of the pile are limited only by the capacity of the handling equipment. A 50-ft. test pile sunk in the sandy soil of the Platte Valley carried 100 tons without settlement.

* Omaha, Nebr.

FLOOD PROBLEMS IN THE ARID REGION

By Arthur P. Davis,* Past-President, Am. Soc. C. E.

A stream of water, whether the smallest rill or the largest river, generally has a definite channel of sufficient capacity to carry its waters in ordinary stages. When these stages are exceeded, some streams overflow their banks, and this gives rise to flood problems. Not always, however, can a stream overflow its banks even in the highest stages.

The Colorado River furnishes an illustration of a stream which can not overflow its banks in some parts, but which can and does do so, at frequent intervals, in others. Throughout the canyon region, where the main stream and its tributaries are rapidly eroding their channels, there is no opportunity for the formation of valleys, and the sides of the canyon are much higher than the summit of the largest floods. In its lower reaches, however, the river emerges from the canyon, and loaded with the sediment collected therein, traverses valleys which have been built of such sediment, and, in ordinary stages, follows its channel to the sea and there deposits its heavy load, building up a delta at its mouth and constantly increasing its length by this means. In order to maintain the grade necessary to carry the sediment, the river must build up its channel at a rate corresponding to the lengthening produced by delta formation at its mouth. This tendency is self-regulating. As the grade of the river decreases, its velocity and capacity for carrying silt decrease at the same time. Therefore, the stream deposits its sediment and builds up its bed, which has the effect of increasing the grade from that point down. It has also the effect of increasing the tendency to overflow and, by such overflow, causing a deposit of sediment in the flood-plain.

Thus, every muddy stream like the Colorado or the Mississippi is increasing its length and building up its flood-plain by overflow. By this process, such rivers form great alluvial valleys attractive to agriculture and, therefore, to population and to the growth of cities. Many of the most fertile valleys of the world are flood-plains of great rivers, which are threatened with a constant tendency to overflow and consequent damage to life and property. In some instances, this tendency is greatly aggravated by geological change. This is true of the eastern plain of China and of the lower reaches of the Amazon, where gradual geologic subsidence has made the middle course of the river lower than it would have been had it been built as a flood-plain without other modifying influences.

In northern latitudes, occasional floods may be caused, mainly by ice gorges, which occur at times of moderate freshet. By obstructing the escape of water, they cause overflow out of proportion to the quantity of water involved. The majority of floods, however, are caused by excessive rain or by sudden melting of large snow accumulations or, frequently, by both these causes. The problem of control includes not only the study of precipitation, but the conditions

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affecting the opportunities for escape of flood waves, including slope and channel capacity. In most cases, the problem is many-sided. All the regions suffering by the floods, under investigation, must be studied, and the remedies proposed should be considered from the standpoint of their influence on all concerned. If a certain city is protected solely by means of levees and channel widening and rectification, such work will have no beneficial influence on the areas below, and it may even slightly increase the difficulty. If detention reservoirs are used, their effect must be carefully considered with respect to all interests down stream.

In general, the benefit of a detention reservoir must be mainly local. The flood problems of Dayton, Ohio, and vicinity have been solved mainly by detention reservoirs, which benefit flood conditions at that point and to a gradually diminishing extent below, but at some point down stream, the benefits of these detention reservoirs disappear or may even become negative. For example, in a widespread storm, the peaks of discharge from different tributaries generally reach the main stream at different times, and detention reservoirs on the stream from which, otherwise, the peak would arrive first, might delay it so as to make it simultaneous with that from the other stream and increase the maximum flow in the main stream. Some such combination is almost sure to happen occasionally in any large complex drainage system provided in part with such reservoirs; but if similar detention reservoirs are provided on both streams so that they are retarded by about equal amounts, this menace is eliminated.

Every great flood, such as that at Dayton in 1913, or that in Pueblo, Colo., in 1921, fixes attention on the immediate locality and necessitates the solution of its problems. At the same time, it calls attention to this menace in other regions, and one may be certain that, as population increases and time goes on, the necessity of solving flood problems will be brought more and more forcibly to public attention.

Storage for flood protection is, to a great extent, antagonistic to storage for other purposes. For irrigation, power, or municipal supply, the proper use of a reservoir generally demands that it be operated in such a way as to keep in storage the largest quantity of water consistent with the necessary draft for use, and these drafts are gradual and more or less regular. Flood prevention, however, demands that, as far as practicable, the reservoirs built for this purpose be held empty so as to receive and store the floods they are designed to control. Whenever a reservoir is used for both purposes there is a tendency for the dominating object to encroach on the domain of the other, and, in general, different reservoirs or distinct capacity in one reservoir, must be devoted to flood prevention and to the various uses required. Notwithstanding these tendencies, almost any large reservoir, even if devoted entirely to industrial uses, will have some influence in regulating flood waves. In many cases, the flood wave may reach a partly filled reservoir and to the extent of the available storage, the flood will be stored and thus controlled. Even if the reservoir is full at the time of maximum flood flow, the surface, if large, will serve as a flood regulator while the accumulating waters raise the level of the lake to the point where the spillway is accomplishing its

maximum discharge. Such a function is performed by every large lake and sometimes constitutes an important regulator on a river system.

Reservoirs provided by the Federal Government primarily for irrigation have sometimes proven important regulators of flood discharge. In 1909, the spring flow of the North Platte River greatly exceeded any previous record. The Pathfinder Reservoir received this flood flow, and, with all outlets opened to full capacity, the reservoir was nearly filled by the time the river reached its maximum discharge. The results of that season's experience indicated that had it not been for the regulative influence of the Pathfinder Reservoir damages in the valley below, by the inundation of farms and towns and the destruction of bridges, would have amounted to nearly as much as the cost of the dam.

Material benefit in flood protection has resulted from the construction of the Roosevelt Reservoir, on the Salt River, Arizona, although it was not designed and is not operated for such a purpose. This is also true, in a less degree, of the Shoshone Reservoir in Northern Wyoming, the Boise Reservoir in Idaho, and others.

One reservoir of the U. S. Reclamation Service—the Elephant Butte Reservoir in New Mexico—has been designed and operated for both purposes, primarily for irrigation, and, secondarily, for flood control. Its capacity is large, and it is not expected to fill frequently, but when it does fill, it is likely to be in a year when considerable water would run to waste. The shallow river bed which extends through the highly improved valleys, overflows easily, and it is desired to restrict the flow to the capacity of the channel, or about 8 000 sec-ft. The drainage below the reservoir, normally dry, sometimes furnishes a flow of this quantity and, at such times, it is proposed to close the reservoir until the peak of the local flood has passed and, at other times, to limit the discharge from the reservoir to a quantity that the river channel can carry with safety. For this purpose, about 400 000 acre-ft. of capacity in the reservoir is reserved. As the water level approaches the spillway lip, it begins to discharge through four wells into tunnels passing under the spillway, and continues to discharge during the rise of the reservoir while filling the 400 000 acre-ft. capacity reserved for flood control. When the spillway begins discharging additional water, the wells will be closed at a rate to hold the discharge to the desired capacity, and as soon as practicable, without exceeding that capacity, the reservoir is emptied to the level of the wells.

The peak of the great flood will seldom coincide with the filling of the reservoir, but when this does occur, it can be operated in the manner described and the discharge can thus be restricted to the safe capacity of the river channel below.

In past years, the floods of the Rio Grande have caused millions of dollars of damage to railroads and other property in its valleys. The damage would greatly increase with the increased settlement and development of the valley under irrigation, if it was not for the protection of the reservoir. With this protection, the damages will be reduced to a comparatively small amount. Such a result, however, depends on the fidelity with which the plan is carried out, by which a part of the reservoir is perpetually reserved for flood control.

The Sacramento River in California is subject to great freshets which cause much damage in its valley. The State of California and the Federal Government are expending large sums in building levees and rectifying the channel. The results desired would be promoted and cheapened if the flood-control efforts could be co-ordinated with the storage works planned, which can be paid for mainly by the irrigation and power interests dependent on them. The storage proposed would not remedy the flood conditions, but it would reduce the peaks of the floods if storage space was reserved for that purpose and works were so operated. However, it has not been possible to co-ordinate the plans of flood control and water storage on this river to the extent that is desirable.

In the lower valleys, watered by the Colorado River, the menace of flood damage is important. The Gulf of California formerly extended northwestward to a point a few miles above the Town of Indio, or about 140 miles from the present head of the Gulf. The Colorado River emptying into the Gulf a short distance south of the International Boundary, carried for centuries its heavy load of silt, gradually building up a great delta cone entirely across the Gulf and cutting off its northern end, which remains as a great depression. Most of the water has evaporated from this depression leaving in its bottom the Salton Sea, 300 sq. miles in extent, with its surface about 250 ft. below sea level. The river flowing over its delta cone steadily deposits silt in its channel and on its banks by overflow, so that it gradually builds up its channel and its banks and forms a ridge, growing higher and higher until the stream becomes so unstable that it breaks its banks in the high-water period and follows some other course. In this manner, in past centuries, the stream has swung back and forth over its delta until this exists as a broad, flat ridge between the Gulf and the Salton Sea, and on the summit of this ridge has formed a small lake, called Volcano Lake, about 30 ft. above sea level. The direct distance from Andrade, where the Colorado River reaches Mexico, to the head of the present Gulf, is about 75 miles, and the distance to the margin of Salton Sea is about the same. As the latter is 250 ft. lower than the Gulf, the slope to the Salton Sea is much greater than to the Gulf of California. This condition, together with the inevitable necessity for such an alluvial stream to leave its channel at intervals, constitutes the menace to the lands lying about the Salton Sea, known as the Imperial Valley. As there is no escape for water from the Salton Sea except by evaporation, the river flowing into this sea, unless diverted, would gradually fill it to sea level, or above, and submerge the cultivated land and the towns of Imperial Valley.

In 1905, the river turned its entire volume into the Salton Basin, eroding a deep gorge and raising the level of Salton Sea. It submerged the salt works and forced the removal of the main line of the Southern Pacific Railroad. At great difficulty and expense, after many unsuccessful attempts, the river was returned to its old channel in February, 1907. The control of the river would be greatly facilitated if the floods were reduced in volume.

The declivity toward Salton Sea is so great that when the river flowed in that direction, it rapidly eroded a deep gorge in the alluvium of Imperial Valley, and, in the process, formed several cascades which gradually retreated

up stream. If this experience should be repeated and the erosion continue for some time, it would cause great damage and the flood of the river in the deep gorge would be difficult to control. It is of first importance that these floods be diminished or abolished.

The most feasible point at which this can be accomplished is in Boulder Canyon at the northwest corner of Arizona, where the river forms the boundary between Arizona and Nevada. At this point, it has been demonstrated as feasible to build a high dam forming a reservoir of almost any desired capacity up to 30 000 000 acre-ft. Such a reservoir would be able to control the entire flow of the Colorado River and permit its utilization for irrigation and power. By keeping 5 000 000 or 6 000 000 acre-ft. in the top of the reservoir as a reserve for flood control and utilizing the remainder available for other purposes, the flow of the Colorado can be confined to less than 25% of its maximum flood discharge.

The fluctuations of this river are such that a large reservoir capacity would be available for flood control coincident with its use for other purposes. The largest floods generally come in May or June, and the low-water season is in the fall and winter. By storing these floods, they would become available for useful purposes by increasing the low-water flow and, at the same time, the flood damage would be eliminated. It is true that the middle part of the basin is subject to floods at other times than the spring, and for this reason the maintenance of the 5 000 000 or 6 000 000 acre-ft. of capacity is necessary to care for such floods. The normal use of the stored waters through the low-water season is sufficient, however, to make available at the opening of the flood season in April, a capacity which generally will be about 10 000 000 or 12 000 000 acre-ft. This capacity will then be available for receiving and regulating the largest floods of the river, which are those that come at this season due to melting snows.

Other storage sites are available farther up in the basin, above the Grand Canyon of the Colorado, but these sites would not be satisfactory for flood control, because they would leave more than 50 000 sq. miles of drainage area uncontrolled.

To make the work complete, it would also be necessary to have a detention reservoir on the Gila River, which occasionally sends a short, flashy flood of destructive volume into the Lower Colorado. These flood waves, however, are so short in duration and so small in total volume that although they menace engineering works, they do not constitute such a material danger to the Imperial Valley as that involved in the long-continued spring freshets of the Colorado.

SOME FACTORS AFFECTING THE PROBLEM OF FLOOD CONTROL

By C. E. Grunsky,* M. Am. Soc. C. E.

Climate and topography introduce peculiar features in the flood-control problems on the Pacific Coast. This will be explained briefly and a few typical projects will be cited, in order to make the underlying principles of flood-control projects clear, rather than to attempt an exhaustive presentation thereof.

The great floods on the Pacific Coast (excluding Alaska from this discussion) occur in the winter or early spring months as the result of heavy rainfall continuing frequently throughout several days. The flood volume of those streams the drainage basins of which extend to high altitudes where there may be much snow on the ground, may be considerably augmented, at times, by the rapid melting of the snow under the influence of what is generally termed a warm rain. As far as is known, however, it appears safe to attribute peak discharges to rain of the maximum possible intensity, as this maximum intensity without any addition of snow may be assumed to be as effective in producing maximum flood conditions as any warm rain that is likely to fall when there is much snow on the ground.

The great cyclonic storms covering vast areas, as they sweep easterly from the Pacific Ocean, are usually accompanied by gentle rainfall continuing for several days. As the storm passes and the wind swings from southeast to southwest, the intensity of the rain increases, it becomes colder, and presently the wind will blow from the northwest and the storm is over. The storm may be followed immediately by others, and it is the heavy rains in these second and third, or later, storms, which are predisposed to produce extreme flood conditions. When a winter season is of the wet type, the rain that falls during these "trailers", finds the ground already saturated and a large part of the rain water must go to the stream. Another noteworthy peculiarity of the Pacific Coast storms is that they are not of the intense type of thunder-storms of the Atlantic Coast and Middle West. Thus, for example, from 2 to 4 in. of rain per hour may be expected on relatively small water-sheds in the East, whereas, on the Pacific Coast, it is rarely that 1 in. per hour is exceeded. Another difference is that there is no ice in the lower reaches of the Pacific Coast streams, and floods due to ice jams do not occur; neither is frozen ground to be taken into account on any of the larger Coast streams.

The drainage basins of the larger streams of the Pacific Coast extend into high mountain regions and parts of these basins may be covered with snow which is compacted more or less by the occasional rains. Therefore, at the close of the wet season, the ground may be covered with 5 or 6 ft. of packed snow. There is shrinkage at times as the season progresses; this shrinkage, however, does not always represent melting, but may be due, in large part, to the compacting that results from rainfall on light snow and from the melting of surface layers of the snow, which forms water that sinks into and aids in compacting the lower layers.

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No definite statement can be made as to the maximum rate at which the snow in the mountains will melt under the influence of the rising temperature and the greater warmth of sunshine when winter is past, but this rate is not productive of the extreme flood conditions of the river. Generally, the rain and snow storms that produce flood conditions, are most frequent in January. They begin to decrease in frequency and duration in February, occur rarely in April, and, after May 1st, no storms are expected that will have any great effect on the flow of the stream in its valley course.

It will be apparent that extreme flood conditions are most likely to occur in midwinter and that the larger rivers of the Coast and those fed by mountain snows will have a long sustained spring and summer high stage. The snow has a beneficial effect in prolonging the time during which the streams are at fair stages. The hydrograph of the San Joaquin and Sacramento Rivers, for example, may show a number of peaks representing the flood stages in winter and then a long sustained moderately high stage, usually terminating in July. The tributaries of these rivers from the east, that is, those which flow into the valley from the snow-capped Sierra Nevada Mountains, behave in a similar manner. This feature makes these streams particularly valuable for irrigation. Even the Colorado and the Columbia partake of similar characteristics when their lower reaches only are considered. Their flow is little influenced by the later summer storms in the high Rockies, which storms are usually of local character and cover little territory.

On Pacific Coast streams, the flow in the late summer and fall dwindles to insignificance, except where a part of the water-shed is porous and the flow is equalized somewhat by the release of water from underground storage. Thus, for example, the Sacramento River, at and below Red Bluff, would go nearly dry in the fall, if it was not for the great volcanic beds of the Mt. Shasta and Mt. Lassen regions, which disgorge water long after the rainy season is past. The flow reaching the Sacramento River throughout the year from such sources is about 4 000 sec-ft., and its principal tributary, Feather River, receives about 500 sec-ft. from similar sources. In its valley section, the San Joaquin River receives a return flow from irrigated lands, of more than 300 sec-ft., and this quantity is expected to increase with the prospective greater use of water for irrigation.

The effect of deforestation on the magnitude and frequency of floods on the Pacific Coast will only be briefly discussed. The timber has been removed in the usual wasteful manner customary in the United States, but this removal has nowhere had any demonstrable effect on the run-off from drainage basins of considerable extent. The clearing of land for farms has been relatively small and, usually, there is so much young timber and other vegetation on the cut-over lands, that the ground has substantially the same degree of protection as under natural conditions. No facts can be cited that will throw any light on the effect of the forests on either the frequency or the magnitude of floods.

The following characteristics of the problems on the Pacific Coast are noted:

First.—The river flows through a valley the surface of which slopes from both sides toward the river. Such is the Columbia River in its lower reaches

and the Sacramento River in the upper part of Sacramento Valley, from Red Bluff to the mouth of Stony Creek, and the San Joaquin River throughout most of its valley course.

Second.—The river debouches on valley land, with a bed slightly depressed below the general surface of the ground, and "fingers" out, sending its flood flow into a number of channels which generally lack permanency and which overflow at moderate, or high stages, sending their waters broadcast over the land toward the main valley drain. Examples of such rivers are the Calaveras, east of Stockton, the rivers of Los Angeles and San Bernardino Counties, California, and a number of lesser streams.

Third.—The river has built up high banks by the deposit of sediment and is flanked by depressions commonly called basins. The Sacramento River, from Stony Creek to Cache Slough, is of this type.

Fourth.—The area to be protected against inundation is a depressed area with no drainage outlet, such as the Tulare Lake region in California and the Salton Basin which extends from California into Mexico.

Where the floods of rivers of the first class are to be controlled, the problem involves either a regulation of the flow by storage reservoirs or protection by levees, of the area adjacent to the stream, that is subject to submersion. Levees will reduce the storage capacity of the flooded area and will force up the flood-plain. The discharge at the peak of the flood will be increased, and suitable provision for adequate channel capacity in the lower reaches of the river must be made. This problem is now being worked out for the San Joaquin River, the particular question being the proper location of the levees—how far shall they be apart—and the increase of channel capacity at the upper end of the river delta. Fortunately, however, in the case of this river, reservoirs of large capacity are being planned on a number of its tributaries the primary use of which is to be for irrigation. These reservoirs, supplementing storage for power purposes already aggregating about 600 000 acre-ft., will have some favorable effect on the flood flow of the main river. Even though all these reservoirs might be full when a great flood comes, they would, nevertheless, in some measure, elongate the flood wave and thereby reduce the peak discharge. Among these reservoirs, may be mentioned:

Pine Flat Reservoir on Kings River	600 000	acre-ft.
Madera Irrigation District Reservoir on San Joaquin River	600 000	" "
Excelsior Reservoir on Merced River	300 000	" "
Don Pedro Reservoir on Tuolumne River	250 000	" "
Melones Reservoir on Stanislaus River	70 000	" "

The Don Pedro Reservoir is under construction and work on the Excelsior Reservoir may begin at an early date. The principal effect of such reservoirs will be to reduce the number of floods. They will probably have comparatively little effect on the maximum discharge of the extreme flood, which is expected only once or twice in a century. The reasonableness of this conclusion will appear when it is recalled that the normal annual water output of the San Joaquin River is about 6 000 000 acre-ft., but that the discharge, in seasons of heavy rainfall, may be 25 000 000 acre-ft., or more.

The flood-control problem on the Pacific Coast has been complicated, as in other regions, by human agencies and activities. For example, the lands that were submersible at high river stages along the Sacramento and San Joaquin Rivers under natural conditions were given by the United States in 1850 to the State of California to be reclaimed. Provision for the disposal of these lands to private parties was made soon thereafter, the first Act fixing the sale price of the land at $1 per acre and placing a limit of 320 acres on the area that a single person could purchase. A few years later, this limit was raised to 640 acres. In 1861, a State Board of Swamp Land Commissioners was created with authority to form districts and to expend $1 per acre on reclamation works. In 1862, the counties were empowered to levy assessments when the cost of reclamation exceeded $1 per acre. By 1865, there had been fifty-four Swamp Land Reclamation Districts formed. In 1866, the powers of the Swamp Land Commissioners were transferred to the county authorities and, two years later, the limit on the area permitted in single ownership was removed. Nearly all the swamp and overflowed land holdings of the State had been disposed of within three years thereafter. In 1874, the limit of area purchasable by a single person was restored to 640 acres.

In 1880, a Board of Drainage Commissioners was created by Legislative enactment but, a year later, the Act creating this Board was declared to be unconstitutional. In 1888, Congress authorized an investigation to determine the damage that had resulted in California to navigable waters by reason of mining operations and, in 1893, it created the California Débris Commission, which outlined the Sacramento Valley control project, now well advanced.

Under the original program of the State, each owner of land subject to overflow was to be allowed to protect himself as best he could. When the lands were sold, no right-of-way reservations were made, as no thought was given to the larger question of bringing the floods under control. Each landowner was permitted to build his own defenses, regardless of the effect of his levees on the stage of water in the river channel or on the spreading of the flood over adjacent lands. Under the resulting system, each landowner or each group of landowners was arrayed against the other; the owners on one side of the stream were pitted against those on the other. Each land-owner tried to outstrip his neighbor on the strength of his levee immediately in front of his property. "If the levee must break, let it be on some other man's land," was the prevailing sentiment. The landowner, living on the bank of the river, which under original conditions was barely submerged at the river's flood stage, soon found himself and his possessions behind a high embankment of earth, softened, perhaps, by long continued rains, with the water of the river steadily rising toward the top of this dike and the menace increasing, until somewhere the dike must give way. It is not surprising that under such circumstances he resented the fact that here and there the head of a swale, or high-water channel, leading away from the main stream, had been closed in the process of land reclamation and that the river had been forced to carry more water than Nature had intended. Thus, it sometimes happened that, at the height of a flood, a dam, closing some such depression, would be cut, and the river would find relief, flooding the lands on the other side.

What should have been done at the beginning was not done until within recent years. There was no comprehensive flood-control project until, in 1908, the California Débris Commission, composed of members of the Engineer Corps of the U. S. Army, recommended the plan that was subsequently approved by both Congress and the State of California. By this time, however, the private attempts at reclamation were extensive. Bank lands, to a large extent, had become highly improved and many settlements, towns, and cities were firmly established on the river front. None of the overflowed land could be re-purchased for rights of way at a price anywhere near that at which the State had sold it, and when it came to providing for a passageway for the water which the river channel could not carry, the owners on each side of the valley wanted the burden thereof placed on the other.

The Sacramento River Flood-Control Project.—In its course through the Sacramento Valley, after passing the mouth of Stony Creek, the Sacramento River presents a peculiar problem. On both sides are flood basins into which, under natural conditions, there was large over-bank flow at every winter freshet and also during the protracted spring high stage. On the west side of the river are the Upper and the Lower Colusa Basins and, farthest down stream, the Yolo Basin. On the east side are Butte Basin, above the Marysville Buttes; then, Sutter Basin which lies between the Sacramento River and its principal branch, the Feather River; then American Basin just above the American River; and, finally, Sacramento Basin, which extends south from the American River into the delta region of the two rivers—the Sacramento and the San Joaquin—the waters of which intermingled at flood on a vast swamp-land area before reaching their common outfall point into Suisun Bay.

The Sacramento River, in its course between basins, occupies a deep channel flanked by narrow strips of high bank land. At some points, the banks are 15 to 20 ft. higher than the land ½ mile to 1 mile back from the river. The river, from a point known as Tisdale Weir, about 26 miles below Colusa, to the mouth of Feather River, about 44 miles, when at flood stage, can carry only about one-eighth of the water—30 000 sec-ft.—as compared with a total of 250 000 to 300 000 sec-ft. This condition which applies, in a somewhat less pronounced degree, to the other valley sections of the river, has led to the adoption of the following basic principles, in planning flood-control works:

(1) Confine to the river all the water that can safely be carried between well located levees of reasonable height.

(2) Allow the water in excess of the river capacity, thus created, to go over the bank at selected points under control.

(3) Prevent the general inundation of lowlands by confining this surplus water between embankments that will conduct it to a re-entry into the lower reaches of the river or into Suisun Bay.

The adopted project based on these principles includes the cutting off of bends and the widening and deepening of parts of the river. The most important channel enlargement of the project is that of the lower reaches of the river. The project has the effect of withdrawing vast areas of valley land from inundation, over which, under natural conditions, the floods could spread,

the great basins acting as retarding reservoirs. Their withdrawal has the effect of raising the river's discharge at the peak of the flood. In this respect, the Sacramento River flood-control project is diametrically opposed to that adopted at Dayton, where artificial retarding basins have been provided for the purpose of reducing the peak discharge.

When the basic principles previously mentioned were first presented by the speaker and his associate, Marsden Manson, M. Am. Soc. C. E., the fundamental consideration of prime interest to the individual landowner and to the State was the reclamation of submersible lands of the Sacramento Valley, almost 1 000 000 acres in extent. These lands were to be protected against overflow. Consequently, the preliminary suggestion did not involve the permanent exclusion of the lands in any basin from the benefit of flood protection. However, when the basic principles were adopted, some of the basins should have been reserved to serve permanently as retarding basins. The advantage of incorporating this feature in the final plan was not recognized and the project is being conducted on the original program. Only in the case of Butte Basin, which has little value as a retarding basin, has it been decided to permit the floods to spread. Sutter Basin is the one basin which obviously could have been of greatest service in reducing the maximum flood flow of the river, but the engineers failed to emphasize this fact and the reclamation of this basin to the total exclusion of flood waters has been permitted.

The flood-plain of the river for a flood stage similar to that of March, 1907,* was assumed, and elevations were fixed, which it is desired shall not be exceeded when there is a recurrence of such a flood. The height of levee tops above this flood-plain was fixed at 3 to 5 ft. Estimates were made of the quantity of water that the river channel can carry at such a flood stage and the places were selected at which relief would have to be afforded by providing outlets. At these points, such as the old Tisdale Weir, already mentioned, at Fremont, near the mouth of Feather River, and near Sacramento, concrete structures have been or will be provided, over the crests of which it is intended that the surplus water shall flow when the river stage approaches the danger line. At extreme floods, it is expected that several hundred thousand second-feet of water will get out of the river channel above Colusa. At the Tisdale Weir, 35 000 sec-ft. will leave the river. This water will be confined between earth dikes and led to a re-entry into the river at its junction with Feather River. As the capacity of the main channel below the Feather River is inadequate, further provision for relief has been made there. Fremont Weir, on the right bank of the river, will be provided, and the water passing over this weir will flow through Yolo Basin between dikes, or levees, which are to be about two miles apart. The areas between these levees have been called "by-passes". The work of constructing the river, the by-pass levees, and the relief weirs is now well advanced, and the flood-control project is nearing completion. Although no definite estimate is available of the effect which the reclamation of the up-river lands will have on the maximum flood discharge of the lower river, provision for greatly increased channel capacity is being made by dredging, which is being done by the Federal

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 281.

Government. The cost of the dredging is divided equally between the United States and the State of California. The capacity of this part of the river is to be about 600 000 sec-ft. This capacity is to be compared with 200 000 to 250 000 sec-ft., which was a peak discharge at high flood stages before natural conditions were disturbed by human activities. In its lower reaches, the river is to be enlarged from an average width of about 1 250 ft. to 3 000 ft. Suction dredges are used for most of this work.

The by-passes are from a few thousand feet to several miles in width. In many seasons, they will receive very little water; there has been hardly more than a bank-full stage of the river for the last three or four years. Even in flood years, if unforeseen breaches in the river or by-pass levees do not occur, the by-pass areas, after a flood stage, will quickly drain and will be available for summer crops.

In 1889-90, as a member of the State Examining Commission on Rivers and Harbors, California, the speaker, in the report of that Commission, stated*:

"The State should modify its reclamation policy. All lowlands in or adjacent to the basins along Sacramento River, together with the delta lands of this stream, should be in one drainage district, and all similarly situated lands along the San Joaquin River should be in another, defined by competent State authority—by a State Board of Public Works—and all work for the improvement of drainage should be planned by such a Board, or under its supervision, and provision should be made to have the work paid for in proportion to benefits resulting therefrom." * * * "This Commission is unanimous in the opinion that * * * these waterways [referring to the Sacramento and San Joaquin River] should, in the interest of navigation, as well as to secure a rapid delivery of flood waters into the Bay, be so treated that they shall flow at their utmost capacity before any water is allowed to escape from them, and that no water should escape from any relief outlet whether natural or artificial, any longer than may be required to prevent a rise of water above a danger line, which should be established from time to time by competent United States or State authority."

After this report was made, more than twenty years elapsed before a comprehensive plan was worked out for the control of the floods of the Sacramento River. This plan was submitted to Congress in 1911 by the Secretary of War on the recommendation of the California Débris Commission. The estimated cost of the project was $33 000 000. Approval was given to the report of the California Débris Commission at a special session of the Legislature of California, in the same year (1911). In 1917, Congress adopted the project and committed the United States to an expenditure of $5 600 000 for the construction of relief weirs, the improvement and rectification of river channels, and the enlargement of the Lower Sacramento River below Cache Slough. The amount named was one-half of that still needed in 1917 for these features of the general project. The State is committed to furnish funds in like amount, and the remainder of the cost is to be provided by the owners of the lands in proportion to the benefits. The total expenditure on the project to date has been more than $20 000 000.

* Report of Examining Commission on Rivers and Harbors, California.

A State Reclamation Board, composed of seven members, not only plans and directs the work, but also has some jurisdiction over private works of reclamation and flood protection, which must be built to conform to the general features of the project.

For the Sacramento relief outlet, the City of Sacramento advanced the necessary funds and took charge of planning and building the structure. It is understood that, for this contribution, proper credit will be given in the final collection of assessments.

The White-Stuck and Puyallup Rivers Flood-Control Projects.—In the State of Washington near Tacoma is a problem with some unusual features. The White-Stuck and the Puyallup Rivers are being brought under control. The White and the Puyallup Rivers, which descend on steep gradients from the mountains to a coastal plain, carry great quantities of detrital matter and drift. The fall decreases where the flat coastal plain is reached and deposits of boulders and drift are likely to occur, which frequently are of such magnitude that the rivers are forced out of their channels. During a flood in November, 1906, the northerly channel of White River was thus clogged with drift and the entire flow was turned south into the Stuck Valley Branch which enters the Puyallup River a few miles above the Town of Puyallup. Engineering studies of this situation have led to the adoption by King and Pierce Counties, of a joint flood-control project, of which W. J. Roberts, M. Am. Soc. C. E., is in charge.

Many engineering reports on this project have been made from time to time, from which a few noteworthy facts have been gained. It was found desirable to hold White River permanently on its new course through Stuck Valley, through which it originally sent only about one-third of its flood flow. The first estimates of channel capacity for flood discharge were too low, and, according to later estimates, a capacity of 65 000 sec-ft. is required. To secure this capacity, the channel of Puyallup River had to be straightened and enlarged, the old channel filled in part, and the banks protected against erosion. The northerly channel of White River was permanently closed by a dam, the concrete face of which had to be protected against undermining. Provision was made, also, for the interception and removal of drift.

Most of the channel excavation was done with a suction dredge which, after the work was finished, was sold to the Imperial Irrigation District where these dredges are used in keeping down the bed load of sand in the Imperial Canal. Various types of bank protection have been tried, several of which have been successful. Among these may be mentioned heavy concrete blocks strung together on wire cable, which blocks are 6 ft. long, 3 ft. wide, and 1 ft. thick. Another successful type of bank protection, used in the tidal sections of the river, was a concrete facing of the bank above low tide with an overlap of the concrete on a brush mattress which was well ballasted with rock or concrete blocks. The concrete bank facing is about 4 in. thick, laid in slabs, 10 ft. wide, separated by strips of wood 1 in. wide.

The device used for the interception of drift consists of a barrier of steel cables supported by thirty large diamond-shaped blocks of concrete, each block weighing about 330 tons. The total length of the barrier is about 2 000

ft., the openings between the concrete blocks being about 45 ft. in the clear, across which ten 1-in. steel cables are stretched. This barrier has functioned quite successfully. After a flood stage, the accumulated drift is removed and burned.

The cost of the work on the White-Stuck and Puyallup Rivers from January, 1914, to the end of 1919, was about $1 630 000.

The Flood Problem of Los Angeles County.—The rivers of Los Angeles and San Bernardino Counties, California, present a problem with somewhat similar features. Only the San Gabriel and the Los Angeles Rivers need be cited as illustrative of the situation. The Los Angeles River breaks from its high mountain water-shed, 170 sq. miles in extent, debouching on a detrital cone which is composed of boulders, cobbles, gravel, and sand—a porous mass over which the waters of the river flow in ill-defined channels, the most significant characteristic of which is lack of permanency. There are no well-defined depressions to give direction to the water, and the channels left by the freshets are broad sand washes with occasional well-defined banks. The gradient of this part of the river's course is relatively steep. Along the ribs of the detrital cone, the fall is generally about 50 ft. per mile. The supply of detrital material seems limited only by the capacity of the river to carry it. When the major floods occur, they flow over the gravel cone at the head of San Fernando Valley, filling old channels and eroding new ones and, on the flatter parts of the valley, the water spreads out to assemble again in a single well-defined channel at Los Angeles, where the river breaks through a range of hills that separates the San Fernando and the San Gabriel Valleys from a broad coastal plain. The floods are then discharged beyond this range of hills on the coastal plain which has been built up, in the course of ages, by the deposit of river silt. On this last lap to the ocean, it is not surprising to find that the river has many times changed its course and has changed also the point at which it has delivered its water into the ocean. Its mouth has shifted between points thirty miles apart. The coastal plain, except along its inland margin at the base of the outlying range of hills previously mentioned, has a flat gradient to the ocean. The river channel across this plain is shallow and unstable, and each recurring flood may mark out a new course to some outfall point miles away from where it had been discharging. During flood stages, the Los Angeles River, together with the San Gabriel, menaces more than 300 sq. miles of the area of Los Angeles County.

This brief description of the Los Angeles River, largely from data assembled by J. B. Lippincott, M. Am. Soc. C. E., applies also to the San Gabriel and the Santa Ana and in a somewhat less pronounced manner to the San Diego River.

When it is recalled that distinctive floods occur about once in nine or ten years, and that, in the intervals, the broad sand washes in the upper valley invite cultivation, it will be seen that these sandy areas could readily be regarded and were treated as areas over which waters ran wild, and their ownership passed into private hands. Old channels and submersible areas have become highly improved farms and are flanked with towns and cities and valuable improvements. In such circumstances, it is not surprising to find

the damage done by the floods of the two rivers in 1914, for example, to have been estimated by Mr. Lippincott at more than $7 500 000, and the incidental depreciation of property values is given by him as possibly $2 500 000 more. In addition, there was great damage to Los Angeles and Long Beach Harbors into which the rivers deposited a large quantity of sand and silt.

In solving the flood problem of Los Angeles County, provision must be made for reducing the flood flow of the Los Angeles and San Gabriel Rivers from their present maximum discharges of about 35 000 and 25 000 sec-ft., respectively, if any material reduction is possible, by storage and by works for spreading the flood flow over broad areas of the upper parts of their previous débris cones and then confining the flood to definite channels with revetted or concrete-faced banks. This work will include a permanent diversion of the flood waters of both rivers from the Harbor of San Pedro (Los Angeles).

The work is being carried out by a district organized under State law. This district is co-extensive with Los Angeles County and its affairs are managed by a Flood Control Commission. About $5 000 000 has already been expended on this project under the direction of Mr. J. W. Reagan, the County Engineer and Chief Engineer of the Flood Control District. Thus far, most of the work has been on the valley part of the rivers, the heavier work in the mountains being deferred on account of the relatively high cost of providing reservoirs for holding back flood waters. It is reported that of the ten or more dams, which are to be constructed to hold back flood waters, one—the Devil's Gate Dam—is nearing completion, and ground has been broken for a second, the Pacoima Canyon Dam, the estimated cost of which is $1 500 000. Two other dams are to be undertaken within the year.

For the main canyon of the San Gabriel River and the Tejunga Wash in the drainage basin of the Los Angeles River, dams are discussed (although they may not yet have been definitely incorporated as project features), which would cost about $15 000 000 and $20 000 000, respectively.

In the official report on the proposed project by Mr. Reagan, submitted in January, 1917, to the Board of Supervisors of Los Angeles County, the required work is referred to as follows:

"The work resolves itself into four parts: First, the construction of both large and small dams in the mountainous areas; second, the protection of the banks of the smaller streams together with spreading and storing of waters of these streams for beneficial use; third, the straightening and rectification through river training and bank protection of the major streams; and, fourth, protection to the harbors and shipping interests."

Then follows a brief enumeration of the principal project features and a statement to the effect that training walls are to be built so that the power of the rivers for good may be utilized in governing and straightening their present tortuous channels:

These training walls will consist of a double row of piling about 5 ft. apart, on the up-stream face of which will be placed wire fencing, and the space between the piling filled with brush and rocks. At the especially dangerous points up and down the streams, bank protection will be used, in order to confine these waters within the single channel, and to prevent the disastrous

wanderings of these streams across the fertile valleys between the mountains and the sea. In general, this bank protection will consist of a double row of piling, or possibly a single row of piling, faced with barbed wire or hog wire, and the space between the two rows of piling, or where the bank is steep, between the single row of piling and the cut bank, filled with brush and weighted down with stones. In all instances, the growth of willows and all trees with a good, deep, firm root system will be encouraged along the banks and behind and in and among this bank protection. On the detritus cone of the San Gabriel River above El Monte there will be excavated a channel 300 to 400 ft. wide and about 4 ft. deep. The material from this excavation will be placed in levees on either side of the stream at distances varying from 600 ft. to ¼ mile, the area between the minor channel and the levees to act as a spreading ground for the waters, that they may be absorbed into underground storage for beneficial use."

A dike is proposed for the diversion of flood waters from the Harbors of Los Angeles and Long Beach.

The Control of Floods at Stockton, Calif.—The Calaveras River has been cited as an example of the second type. The City of Stockton, Calif., lies in the heart of the area, which was subject to frequent submersion by the flood waters of Calaveras River. This river is a small stream with 491 sq. miles of low mountain water-shed and is practically dry in the summer and fall. Under natural conditions, the river, near the point where it reaches the easterly margin of the San Joaquin Valley, spilled its floods over its banks and these waters flowed westerly, following natural swales and depressions, to an outfall in the swamp-land areas westerly from Stockton. These waters had cut channels which, in their lower reaches, were of large capacity, but which, in many cases, "fingered" out to mere swales at their heads some distance east from Stockton. There was a junction of several of these high-water channels in Stockton, forming Mormon and Stockton Sloughs, the lower reaches of which are at sea level and have always been navigable. Because the annually recurring high waters brought down much silt which became an obstruction to navigation in the Stockton Channel and increased the maintenance charges against this waterway, the Federal Government undertook the construction of a diverting canal, the right of way and the general plan of which were provided by the City of Stockton. This diverting canal was given an oblique position across the natural slope of the country, and the excavated material was deposited on its westerly or down-stream bank. Its original capacity was about 7 000 sec-ft. The flood flow to be diverted is four or five times this quantity; consequently, this canal, although affording protection to lands at the west, has had little, if any, effect in reducing the inundation of lands to the east. It is even claimed by some that matters have been made worse instead of better, particularly as the canal does not appear to have sufficient grade to give the water a scouring velocity and silt deposits are reducing its capacity.

It is known that, in the mountains, there are reservoir sites of ample capacity to reduce the flood flow so that it can readily be conducted to its outfall into the San Joaquin River in existing or slightly modified channels. Obviously, reservoir control of the floods should here be made a project feature.

The Flood Problem at Tulare Lake.—The Tulare Lake region and the Salton Basin have been referred to as illustrative of projects of the fourth

type. With Tulare Lake, an area about 750 to 800 sq. miles in extent comes into consideration. After the great flood year of 1861-62, this lake reached a very high stage. It fell somewhat in the succeeding years, but again rose in 1867-68 to about the former level, which is generally accepted as its highest known stage. A gradual recession of the lake water has taken place since and there has been little or no northward outflow from the lake since 1876 or 1877. In 1898, the lake went dry and has been dry again in a number of seasons since 1904. Much of the land laid bare by the receding waters is very fertile, and farming has become quite extensive and profitable. The risk of a recurrence of high stages in the lake has been considered to be so slight that permanent improvements have been built as though a return of such wet conditions as are known to have occurred within the memory of man, would never happen again. Railroad lines and towns are located on land that at one time was well within the lake.

Tributary to this depressed area are the water-sheds of the Kern, Tule and Kaweah Rivers and, partly or periodically, Kings River, together with a number of lesser drainages and quite an extent of Coast Range hills on which, however, the rainfall has always been very light.

The barrier across the "thalweg" of the San Joaquin Valley, to which is due the Tulare Lake depression, was built by Kings River. If unrestrained by levees this river would send part of its flood flow south into the lake and part of it north to join the waters of the San Joaquin River. Regardless of what disposition is made of the discharge of Kings River, the flood menace to the Tulare Basin will remain. No adequate provision has been made for the interception and by-passing of the flood waters from the other rivers. All these streams are allowed to reach the lowest depression, and the attempt has been made to hold the water in check as it rises under the influence of rains and melting snows by means of concentric levees. In seasons of light rainfall, all the levees may hold, but, in flood years, one line of levee after another breaks, and the crops of a year or two on much of the lake bed may be lost. The greatest safety lies well up toward the old high-water line, but, unfortunately, these lands, as a rule, are the least fertile.

The conviction seems to be quite general that natural conditions, due mainly to the extensive use of water for irrigation, have been changed so that Tulare Lake will never again rise to its one-time high stages, and consolation is found by some in the fact that since 1889-90 there has been no season in which the rainfall approached this region's probable maximum of twice the normal. The unwelcome truth should be recognized, however, that irrigation, when the next very wet year comes, will not materially reduce the volume of flow into Tulare Lake, and there is no sound basis to the opinion that the climate has changed and that there will be no more extremely wet years. On the northeastern margin of the lake, as it was in the fall of 1882—about 20 ft. below extreme high-water mark—the speaker sketched a group of willow stumps, one hundred or more in number, some of which were 4 ft. in diameter. It would not have been possible for trees to have attained that growth if the lake had not been at a low stage for many years at some time preceding 1853, its history being known since that date. Indian tradition confirms this,

referring to a time when the lake had contracted to two small ponds, with dry land between. This period of low water must have been followed by a series of years when the lake was at a moderate elevation. The trees died, and the wood above water decayed, leaving the jagged ends of their stumps from 3 to 4 ft. above the surface of the ground. These stumps which, no doubt, have long ago succumbed to farming operations, were located on the south bank of the Mussel Slough, one of the original high-water branches of Kings River.

There is, therefore, no comprehensive plan for the permanent reclamation of Tulare Lake lands, and not much is likely to be done in securing such a plan until a disastrous flood emphasizes the menace.

The Flood Control Problem of Lower Colorado River.—The Salton Basin, of which the southern part is known as Imperial Valley, was, at certain prehistoric times, the outfall point of Colorado River. This basin is the depression below sea level, which, in the course of ages, has been cut off from the upper or northern end of the Gulf of Lower California. At one time the Gulf extended to and beyond Indio, Calif., a station in Coachella Valley on the Southern Pacific Railroad. The silt delivered by the Colorado River into the Gulf has built up a broad delta cone that has its apex near Yuma, Ariz., at an elevation of about 120 ft. above sea level, and that extends, with a broad, flat crest, across the valley in a southwesterly direction, about 50 miles to the Cocopah Mountains.

A sediment ridge, flat to the eye, has thus been built, that separates the Salton Basin from the Gulf. The lowest point of this ridge is about 35 ft. above sea level and it is at this elevation that an old beach line is readily traceable around the Salton Basin. About 2 000 sq. miles of the Imperial and Coachella Valleys lie below this old beach line. The lowest point of the basin is about 287 ft. below sea level, and all the cultivated areas in the valleys are below sea level.

Evidence is unmistakable that, in prehistoric times, the Colorado River has swung back and forth on its delta cone, discharging for a time into the Salton Basin and, again, into the Gulf, just as the Amu Darya, a river of Asia—the Oxus of ancient history—periodically flows into the Aral Sea and, again, into the Caspian Sea.

Due to early explorations of the Spaniards, it is known that, for 500 years, and probably much longer, the Colorado River has held to its course into the Gulf of California, except a little more than a year (1905 to 1907), during which it took a course to the north and filled the Salton Basin to a depth of 80 ft.

Just north of the point where the southern boundary of California reaches the river, water has been diverted from the Colorado River, during the last 25 years, for the irrigation of lands in the Imperial Valley. A canal of large capacity was required and the erosive action of the water was the main agency of first construction. In later years, dredges have been in use in bettering the alignment of the canal and in building up its banks. The summer flow of this canal is about 6 000 sec-ft. The canal takes a course through Mexico, this course being made necessary by topographic features. The area under

irrigation with water from this canal may be given, in round numbers, as 420 000 acres in the Imperial Valley, California, and 150 000 acres in Mexico.

The silt carried by the Colorado River amounts to about 0.8% by weight or about 112 000 acre-ft. of compacted material per year. The banks of the river in their natural condition were densely overgrown with cottonwood and willow trees, weeds, and grass, so that the annual overbank flow, in the delta reaches of the river, was obstructed and had no tendency to erode the ground and form pronounced high-water channels until it was at some distance from the main stream. The permanency of the river on its original course was thus fairly well assured. An artificial channel so concentrated the outgoing water that it commenced to cut a channel. When this was once started, it could not be stopped, and in 1905 a small dredger cut, made in 1904, became the main river channel, and for a year the river poured its water into Salton Basin. With great difficulty and at large expense, operating on Mexican territory, the river was turned back into its natural channel in December, 1906, by the Imperial Irrigation District, only to break out again a few weeks later. It was then successfully turned back through the agency of the Southern Pacific Company. By this time, the whole aspect of the delta region had changed. Brush and grass had died and fires had cleared the ground so that thereafter overbank flow was less impeded than it had been previously. In 1909, the inevitable happened; the river broke through its west bank, in Mexico, about opposite the southern boundary line of Arizona, and appropriated and enlarged the overflow channel known as the Rio Abejas, or Bee River, in which it flowed westerly into Volcano Lake and thence in various channels and across country toward the south, with its outfall into the so-called Hardy Colorado.

It was necessary to build a levee just north of Volcano Lake to keep the water from taking a northerly course. Under natural conditions, outlets from this lake carried the water as well to the north as to the south. A low levee had been constructed there, in 1908, as the lake had already commenced to show its tendency to overflow the low, flat bank lands. The silt and drift which the Colorado River has now been unloading in the Volcano Lake region, have warped this area and have brought the flood-plain up at an average rate of practically 1 ft. per year. Levee building has kept pace with this rise, so that at present the Volcano Lake levee has a height of about 14 ft., a railroad track on its top, and its southerly or water face is heavily rip-rapped. Much of the material of which this levee is built was hauled in by train, because of the fact that the western 8 or 10 miles of levee rests on ground too alkaline for use in levee building.

About $5 000 000 has been expended on this flood-control work, this sum having been contributed mainly by the Imperial Irrigation District of California, but expended in Mexico.

Desultory attempts are now being made to coax the river into a location more to the south of its present course, but even if these attempts should be successful, or partly so, they will not solve the flood problem of the river. There is only one course to be taken; the river must be placed on a direct course to the Gulf, not necessarily by way of the old channel, which is tortuous

and lacks capacity, but on some location fairly parallel thereto. Then, as a matter of equal importance, the number of floods and particularly the long sustained summer high stages of the river should be reduced by the construction of a reservoir of large capacity somewhere on the lower river. The Boulder Canyon site has been suggested for this purpose, and studies of the feasibility of a high dam in that canyon have been made by the U. S. Reclamation Service.

The Boulder Canyon Dam site is just above the point where the Colorado River makes its abrupt turn from a westerly to a southerly course and is 3 or 4 miles below the mouth of the Virgin River. The canyon is reported as about 250 ft. wide for a distance of ½ mile. The sides of the gorge are steep, and the rock is granite. The project for storage at this site, as now favored by the U. S. Reclamation Service, involves the construction of a dam that would be about 550 ft. in height above the present low-water surface of the river. The storage capacity of the reservoir created by such a dam would be more than 25 000 000 acre-ft. The annual discharge of the Colorado River at this point ranges from about 7 000 000 to 22 000 000 acre-ft., and its normal annual discharge is about 15 000 000 acre-ft. A large part of the storage space would be in the lower valley of the Virgin River. The water of a full reservoir would extend up stream to the lower end of the Grand Canyon. The reservoir could be manipulated so that it would eliminate the lower-river flood menace from the up-river high stages. It would regulate the flow of the river for irrigation purposes and would justify the installation of a power plant with an output capacity of about 700 000 h. p.

This reservoir would completely remove the flood menace to the Colorado River lands at Palo Verde and at other points above the mouth of the Gila. It would also prevent all extreme flood stages below the Gila, except those of infrequent occurrence caused by the Gila itself. The Boulder Canyon Reservoir, together with the outfall correction into the Gulf of California, would adequately solve the flood-control problem of the Colorado River.

The cost of providing a reservoir at the Boulder Canyon site is generally given as about $50 000 000. The agency which should carry out the regulation of the flow of the Lower Colorado River in the interest of irrigation, for the development of power and for flood control, is the Federal Government which is the only agency that can speedily carry out the rectification of the alignment of the river in Mexico. There should be some understanding reached at once with Mexico, under which the United States will be permitted to do this work and to maintain any newly established outfall channel, and this understanding might well be coupled with some arrangement for a delivery to Mexico of some of the water to be stored at Boulder Canyon, subject, however, to a proper adjustment of the costs of the work according to the benefits conferred.

FLOOD CONTROL IN THE MIAMI VALLEY, OHIO

By Charles H. Paul,* M. Am. Soc. C. E.

The flood of March, 1913, was the greatest ever recorded in the Miami Valley. Large sections of the business and residential districts of Dayton and Hamilton, Ohio, were inundated to depths of 10 to 12 ft. At Dayton, the levees guarding the down-town district were overtopped 6 ft. or more, and the peak flow through the city exceeded the capacity of the river channel nearly 300 per cent. Similar conditions prevailed in other cities and towns in the Valley. The property loss was estimated at $100 000 000.

Relief committees were organized in the different cities immediately after the flood and plans for the prevention of future floods began to be formulated. Each community worked independently at first, the thought being that the desired results could be accomplished by channel enlargement. Soon after engineering study was begun, however, it became apparent: First, that channel enlargement alone, to the extent required, was not practicable; and, second, that the people of one city, alone, could not adequately protect themselves, and that it was a job for the people of the whole Valley to undertake as a unit. A combination of channel improvement and retarding basins was the final solution of the problem.

Under the Ohio laws, there was no practical way for the people of the Valley to organize as a unit, and it became necessary, therefore, to enact legislation for that purpose. The Conservancy Act of Ohio, passed in 1914, paved the way for the organization of the Miami Conservancy District, which was effected soon thereafter.

This Act provided that a complete outline of the proposed work, known as the "Official Plan", including plans, specifications, and estimates of cost, be prepared by the Directors of the District and submitted to the Conservancy Court for approval. The Conservancy Court for the Miami District is composed of one Common Pleas Judge from each of the nine counties affected. Immediately after the organization of the District, work was begun on the preparation of the Official Plan, and a large engineering force was employed for more than a year getting it ready to be submitted to the Court. The case was then set for hearing, at which any interested person could appear and make objection to the plan as a whole, or suggest changes in any particular. Some opposition had developed, particularly among the owners of lands that would be required for the retarding basins, and in some communities, where there was a feeling that local treatment other than that set forth in the Official Plan, would be more economical. This resulted in a thorough scrutiny of the Plan during the hearing, and it was approved by the Court without any material changes.

The next step was an appraisal of damages or benefits to property affected by the proposed construction. A Board of Appraisers, appointed by the Court, prepared an appraisal roll of damages and benefits, which was presented

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to the Conservancy Court for approval. Any property owner who was not satisfied with the award of the appraisers had the opportunity of being heard by the Court. The Roll was approved with so few exceptions that the security for bonds, as represented by uncontested appraisals, was ample to justify proceeding with the financing of the project and preparing for the construction of the works.

Fig. 33 shows three main branches of the river coming together at Dayton. From west to east, they are the Stillwater, Miami, and Mad Rivers, flowing south under the name of the Miami. Wolf Creek joins the Miami also at Dayton, just below the junction of the three main tributaries. Loramie Creek flows into the Miami at the upper end of the Valley, just above Piqua. Twin Creek joins the main river between Franklin and Middletown, and Four-Mile Creek comes in just above Hamilton. The drainage area above Hamilton is 3 600 sq. miles, and that above Dayton comprises 2 600 sq. miles.

The Miami Conservancy Plan, as finally adopted, provides for channel improvement through all the cities and towns affected by floods, to the extent that is economically feasible, supplemented by retarding basins which will hold back the crest of the flood, by restricting the flow through the dams and backing up the surplus water temporarily, in the basins. The outlets through the dams are not controlled by gates, but are designed of such size that their combined discharge, under maximum head (full basin), will not overtax the capacity of the improved river channels through the cities below. Provision is made for a maximum flood 40% greater than the 1913 flood. It is believed that this maximum is in excess of any flood that can occur in this locality. Channel improvement is provided at Piqua, Troy, Dayton, West Carrollton, Miamisburg, Franklin, Middletown and Hamilton. (See Fig. 33.) Dams which form retarding basins, are located at Lockington on Loramie Creek, at Englewood on Stillwater River, at Taylorsville on the Miami, at Huffman on Mad River, and at Germantown on Twin Creek. The cost of the work is met by special assessment against the property benefited.

Organization

It was expected to have the construction work done by contract and the principal divisions of work were advertised for bids in the fall of 1917. The United States, in the meantime, had entered the World War, labor and industrial conditions had become acute, and it was realized that, in justice to himself, no reliable contractor could make a reasonable bid on jobs as large as this, which would require four or five years to complete. Bids were opened in November, 1917, and with the exception of one bid for a small section of channel improvement, all of them were either unreasonably high or were irregular.

The Directors of the District were then faced with the postponing of construction, or of perfecting a construction force as a District organization, purchasing equipment, and proceeding with the work. The latter course was taken and results have shown the wisdom of that decision.

The engineering forces were already well organized. A construction manager was selected and superintendents were secured for the larger features of

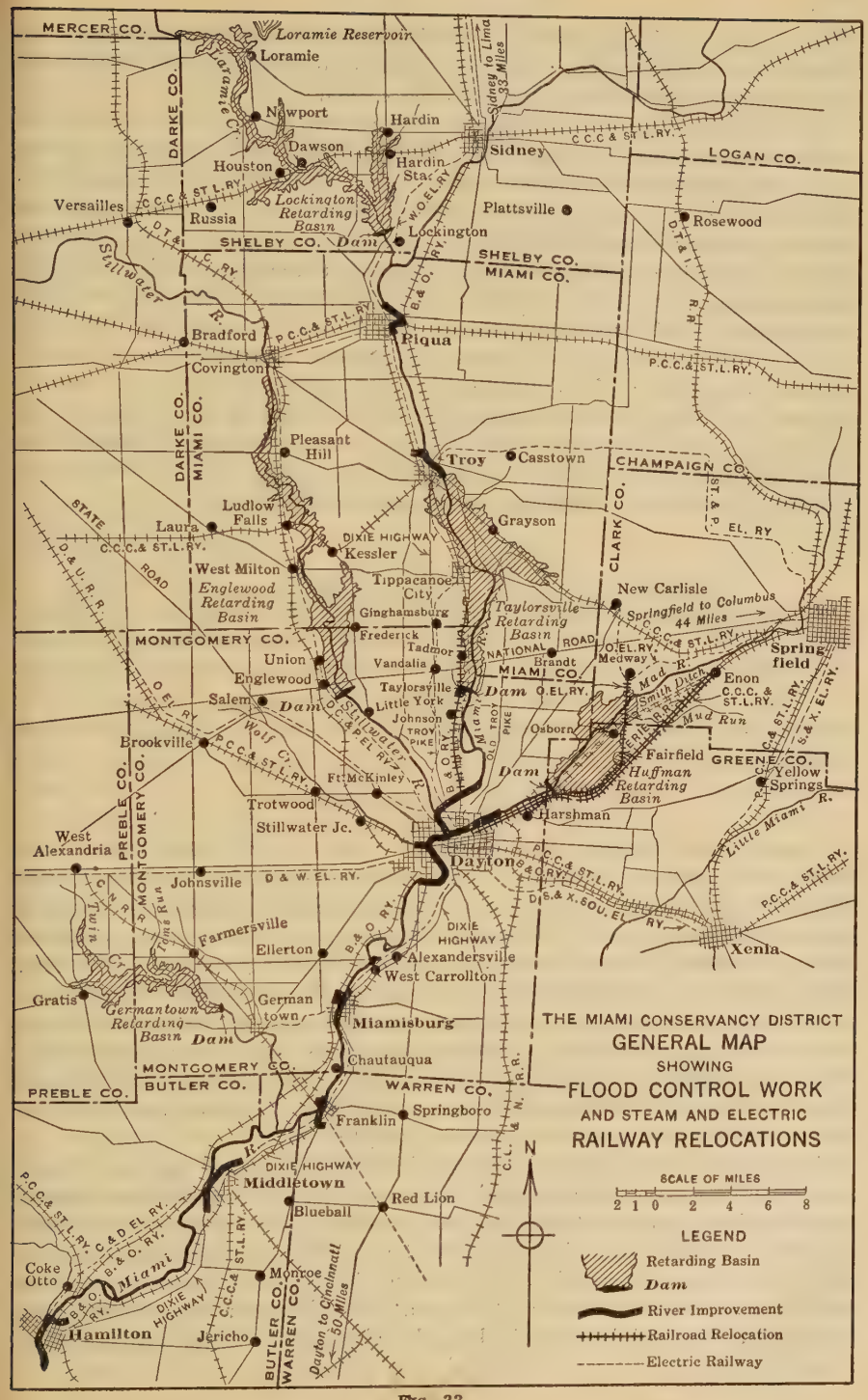
MERCER CO.
Loramie Reservoir
Loramie
Loramie Creek
DARKE CO.
Newport
Hardin
Sidney to Lima 33 Miles
Dawson
Houston
Hardin Sta.
Sidney
C.C.C. & St.L.Ry.
LOGAN CO.
Versailles
Russia
Lockington Retarding Basin
T.W.O.El.Ry.
Plattsville
Rosewood
O.T. & C.Ry.
SHELBY CO.
Dam
Lockington
SHELBY CO.
MIAMI CO.
Stillwater R.
B. & O. Ry.
Bradford
P.C.C. & St.L.Ry.
Piqua
Miami R.
Covington
P.C.C. & St.L.Ry.
DARKE CO.
MIAMI CO.
Pleasant Hill
Troy
Casstown
CHAMPAIGN CO.
Ludlow Falls
Laura
C.C.C. & St.L.Ry.
DIXIE HIGHWAY
Kessler
Grayson
CLARK CO.
C.St.L. & P. El.Ry.
STATE ROAD
West Milton
Englewood Retarding Basin
Tippecanoe City
Ginghamsburg
Taylorsville Retarding Basin
New Carlisle
Springfield to Columbus 44 Miles
MONTGOMERY CO.
Frederick
Tadmor
NATIONAL ROAD
O.El.Ry.
C.C.C. & St.L.Ry.
Springfield
Union
Vandalia
Brandt
Medway
Mad R.
Englewood
MIAMI CO.
Taylorsville
Dam
O.El.Ry.
Enon
C.C.C. & St.L.Ry.
Salem
Dam
Little York
Johnson
TROY PIKE
Miami
OLD TROY PIKE
Smith Ditch
Mad Run
Osborn
Brookville
Wolf Cr.
P.C.C. & St.L.Ry.
Ft. McKinley
Fairfield
GREENE CO.
Yellow Springs
Huffman Retarding Basin
Dam
Trotwood
Harshman
PREBLE CO.
MONTGOMERY CO.
Stillwater Jc.
West Alexandria
Dayton
P.C.C. & St.L.Ry.
D. & W.El.Ry.
Johnsville
D. & X. Sou.El.Ry.
Little Miami R.
P.C.C. & St.L.Ry.
Farmersville
Ellerton
B. & O. Ry.
DIXIE HIGHWAY
Alexandersville
West Carrollton
Xenia
Gratis
German town
Germantown Retarding Basin
Dam
Miamisburg
MONTGOMERY CO.
BUTLER CO.
Chautauqua
WARREN CO.
PREBLE CO.
Springboro
Franklin
C.L. & N. R.R.
DIXIE HIGHWAY
Middletown
Red Lion
Blueball
P.C.C. & St.L.Ry.
C. & D.El.Ry.
B. & O. Ry.
Coke Otto
Miami
Monroe
C.C.C. & St.L.Ry.
DIXIE HIGHWAY
Hamilton
Jericho
BUTLER CO.
WARREN CO.
Dayton to Cincinnati 50 Miles
N
THE MIAMI CONSERVANCY DISTRICT
GENERAL MAP
SHOWING
FLOOD CONTROL WORK
AND STEAM AND ELECTRIC
RAILWAY RELOCATIONS
SCALE OF MILES
2 1 0 2 4 6 8
LEGEND
Retarding Basin
Dam
River Improvement
Railroad Relocation
Electric Railway

FIG. 33.

the work (the five dams, and the channel work at Dayton and Hamilton). In the spring of 1918, enough of the construction organization and equipment had been assembled so that work could be started.

The general plan of organization for construction was to maintain, as nearly as practicable, the relationship between engineering and construction forces that would have existed on contract work. On each main feature of the work was a division engineer, with authority such as he would have had on a contract job, and a superintendent who had direct charge of the construction forces. The superintendents reported directly to the construction manager whose relation to the job in many respects was like that of a contractor. The specifications which had been prepared in anticipation of letting contracts, were followed, with such modifications as the change in organization demanded. Although responsibility for the quality of work, sufficiency of methods, rate of progress, and costs, were allocated much as they would be on contract work, there was a sense of co-partnership between the engineering and construction offices, which made for better results, more rapid progress, and lower costs, than would have been possible on an ordinary contract job.

Railroad Changes

It was necessary to re-locate 50 miles of railroad lines throughout the Miami Valley, in order to get them out of the way of the dams and retarding basins. New locations satisfactory to the railroad companies were made at the expense of the District, under the general direction of the railroad companies. The old lines then became the property of the District, the rails and the ties being salvaged.

Channel Improvement

Channel improvement is almost certain to be an economical feature of a flood-control project. Even when full protection cannot be secured by such means, there is usually much to be gained, at moderate expense, by removing bars and islands, and by deepening, widening, or straightening channels within reasonable limits. The cost of additional capacity increases relatively, until finally an economical limit is reached. In the Miami Conservancy Project, the retarding-basin control has permitted channel enlargement to be confined to moderate limits through most of the cities, except Hamilton, where a considerable widening of the channel, which had been severely encroached upon by industrial plants, was unavoidable. The work at Hamilton involved considerable property damage. (See Fig. 34.)

Three general methods of channel improvement were required: First, that in which channel excavation is the essential or most prominent feature, as at Hamilton and Dayton; second, that in which the work is confined almost entirely to levee construction, as at West Carrollton, Miamisburg, Franklin, and Middletown; and, third, a combination of these two features, as at Troy and Piqua. The situation, at Troy and at Middletown, was improved by cut-off channels and in several places by clearing out trees and other obstructions.

The standard section, Fig. 34, at Dayton and Hamilton has a low-water channel about 150 ft. wide, with flat slopes, or beaches, on either side, extending out to the toes of the levees. Where the river is straight, the low-water channel is located in the center and where the channel is curved, near the outside of the bends. Having determined the channel capacity obtainable at reasonable expense, the object then was to design a standard channel section to give velocities of flow as nearly uniform as possible at any given stage, and one that will be reasonably self-maintaining. It was necessary, in some cases, to narrow the old channel in order to prevent a decrease of velocity, which would result in the formation of bars.

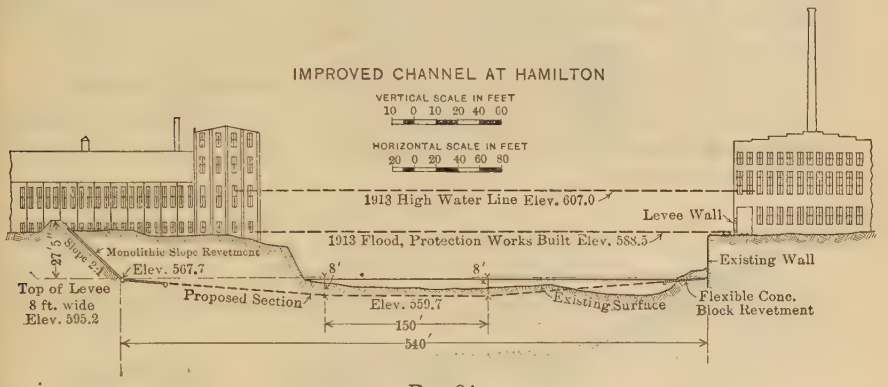

FIG. 34.

Channel improvement in most cases resulted in increased velocities of flow. To prevent erosion of banks where good sod is not sufficient, concrete revetment was used. (See Fig. 34.) On the levee slopes this revetment is of monolithic slabs, 6 in. thick, reinforced with wire mesh. These slabs are divided by expansion joints into sections of about 8 by 12 ft., the long dimension being up and down the slope. On the flatter slopes, along the toe of the levee, the revetment is a flexible mat of concrete blocks, 2 ft. long, 12 in. wide, and 5 in. thick, strung on ½-in. galvanized wire strand. Fig. 35 shows details of this block revetment. Concrete retaining walls were used in many places, where the width of the channel is so restricted that levee slopes would require the taking of expensive properties.

Dragline excavators were used on all the larger features of the work, not only in making the channel excavation through the cities, but also in excavating for the outlet structures at the dams, and in making the borrow-pit excavation for the embankment material at the dams. The District used twenty-one of these machines varying in size from the small machine having a 30-ft. boom and a ¾-cu. yd. bucket, to the large machine with a 100-ft. boom and a 5-cu. yd. bucket. In many places, the dragline placed the material excavated from the channel directly in the levee. Where this could not be done, it was sometimes economical to move the material two or even three times to the levee.

At Dayton and Hamilton, however, the channel excavation was so much in excess of levee requirements, that a large part of the excavated material

had to be wasted. The nature of the channel at Hamilton was such that a construction track could be placed within the river out of danger from moderate floods. The waste material was loaded into 12-cu. yd. dump cars and hauled to the waste banks by 40-ton, standard gauge, dinkey locomotives.

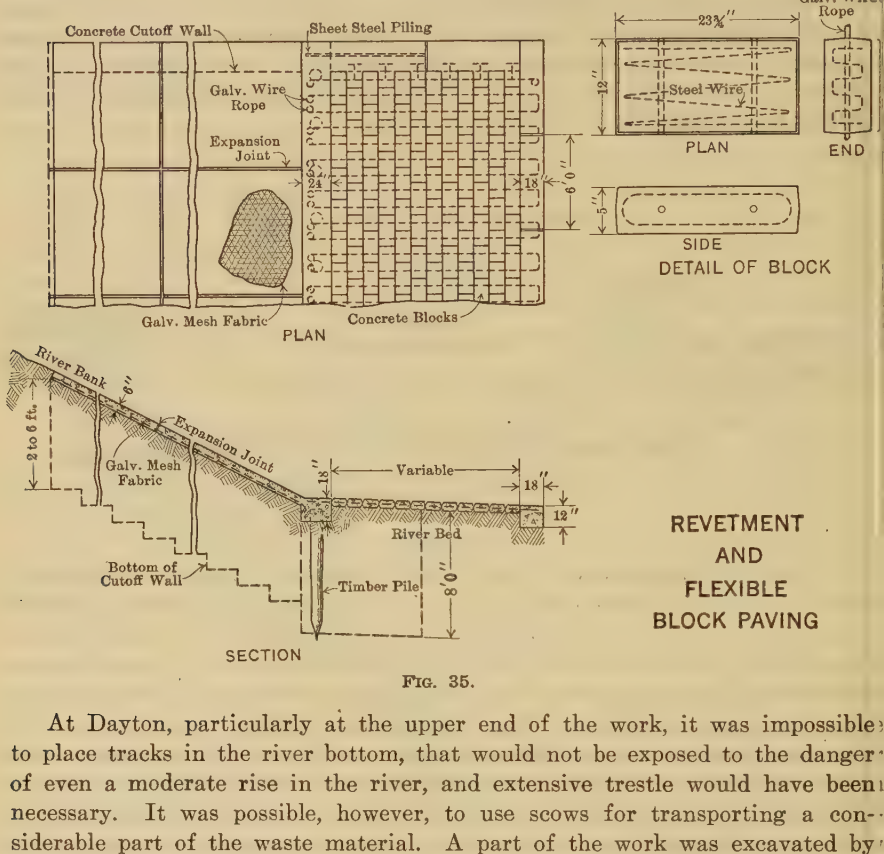

FIG. 35.

At Dayton, particularly at the upper end of the work, it was impossible to place tracks in the river bottom, that would not be exposed to the danger of even a moderate rise in the river, and extensive trestle would have been necessary. It was possible, however, to use scows for transporting a considerable part of the waste material. A part of the work was excavated by one of the large dragline machines placed on a scow. The loaded scows were towed to an unloading point near the waste bank where they were unloaded by another dragline machine, which placed the material directly into the spoil bank. At the other cities, where levee construction was one of the principal items of work, the levees were usually built by dragline machines, the necessary material being obtained from borrow-pits or from channel excavation, whichever was most practicable or economical. In a few cases, it was more economical to build sections of the levees by teams.

THE DAMS

One of the big engineering problems was the determination as to number and size of retarding basins, and the balancing of retarding basin capacities, outlet capacities through the dams, and improved river-channel capacities

through the cities. The details of this problem have been discussed in one of the Technical Reports published by the District.* The project included the construction of five dams at the locations shown in Fig. 33. The principal dimensions of the five dams are given in Table 4.

TABLE 4—Size of Miami Conservancy Dams.

	Germantown Dam.	Englewood Dam.	Lockington Dam.	Taylorsville Dam.	Huffman Dam.
Maximum height, in feet.....	110	125	78	78	73
Length, in feet.....	1 200	4 700	6 400	3 000	3 300
Earthwork, in cubic yards.....	800 000	3 600 000	970 000	1 130 000	1 350 000
Concrete work, in cubic yards..	17 400	26 500	32 000	48 000	37 500

Foundation conditions dictated the selection of earth dams at all the sites. Borrow-pit investigations indicated that hydraulic-fill dams would be practicable and economical, and it was decided to build at each site a hydraulic-

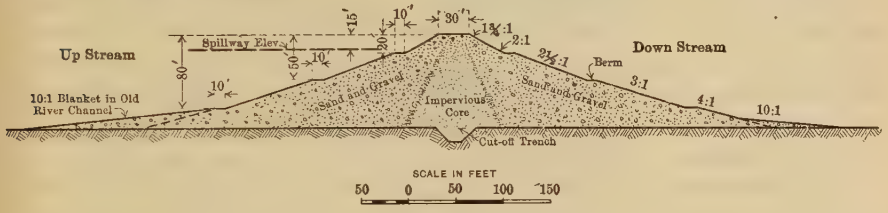

Fig. 36.

fill dam with concrete outlet structures designed to give the required control of floods. Fig. 36 shows the standard cross-section adopted for all the dams. The core width is an important feature of this design, as there are practical objections to a core that is either too wide or too narrow. The wide core, extending out under the slope material, gives scant support during the early stages of consolidation, and thus encourages the sloughing of the slope material. The narrow core permits tongues of coarse porous slope material,

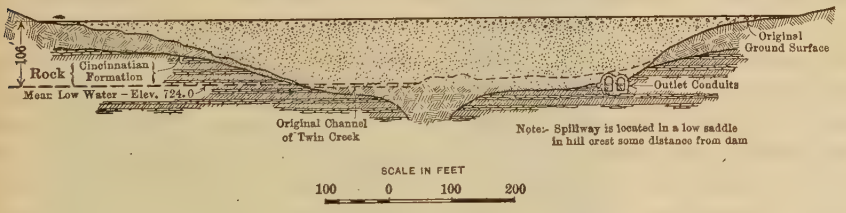
Original Ground Surface
Rock
Cincinnatian Formation
Outlet Conduits
Mean Low Water – Elev. 724.0
Original Channel of Twin Creek
Note:- Spillway is located in a low saddle in hill crest some distance from dam
SCALE IN FEET
100 0 100 200

Fig. 37.

which frequently extend into or across the core zone, or far enough to meet similar encroachments from the other side, thus resulting in a porous section through the core. In the design adopted, which proved satisfactory, the theoretical core width at any point is equal to the height of dam above that

* Technical Reports, Vol. VII, "Hydraulics of the Miami Flood Control Project."

point. Fig. 37 shows a longitudinal section of the axis of the Germantown Dam, with the outlet conduits on the rock foundation at one side of the old river channel. The spillway at this dam is in a low saddle in the hill crest, some distance away.

Outlet Works.—The first important item of construction at each dam was the building of the outlet structure, or at least enough of it so that the river could be diverted from its old channel to permit the construction of the dam. Suitable rock foundations for the outlet and spillway structures were found at all the dam sites. Two types of outlet structures were used. One type, which was used at three of the dams (Lockington, Taylorsville, and Huffman), was an opening through the dam formed by two retaining walls with a concrete floor between. This opening gave ample waterway for floods during the construction period. After the dam had been built to the point where there was no further danger of overtopping in case of a flood, a cross-dam was built between the two retaining walls, and through the bottom of this cross-dam, at river level, the conduits for permanent flood control were constructed. The top of the cross-dam was placed 12 to 15 ft. below the top of the dam, thus forming the spillway. Fig. 38 shows a typical structure of this type. This design has two advantages: First, no additional expense is required to obtain a waterway large enough to care for floods during construction; and, second, the conduits and spillway are combined in the same structure.

At Germantown and Englewood, however, the dams are so high that the cost of retaining walls to sustain the hydraulic-fill embankments was prohibitive, and two conduits, at the elevation of the old river bed, extending through the base of the dam, were used. In order to secure additional capacity to care for floods during construction and prevent the overtopping of uncompleted embankment, these conduits were made deeper than was required. As soon as the dam had reached a height so that there was no further danger of being overtopped by floods, the bottoms of these conduits were filled in and floored over, leaving the conduit opening of the size required for permanent flood control. Fig. 39 shows some of the details of this type of conduit, particularly the additional capacity available for use during the construction period, and the method of reducing the size of opening to that needed for permanent flood control. It also shows the location of construction joints in the conduit arches, the interesting feature of which is the pair of joints near the crown of the arch. The "keystone" section between those two joints was poured after the concrete on either side had set, and this method proved to be effective in preventing cracks along the crown of the arch. At the two dams (Germantown and Englewood), where these conduits were used, the spillways were separate structures.

At the down-stream end of each of the outlet structures, a stilling pool was built for the purpose of dissipating the energy of this swiftly flowing water. The details of this stilling pool are shown on Fig. 40, which is a longitudinal section through the conduit and outlet structure of the Germantown Dam. Each structure is designed so that the hydraulic jump, or standing wave, will be

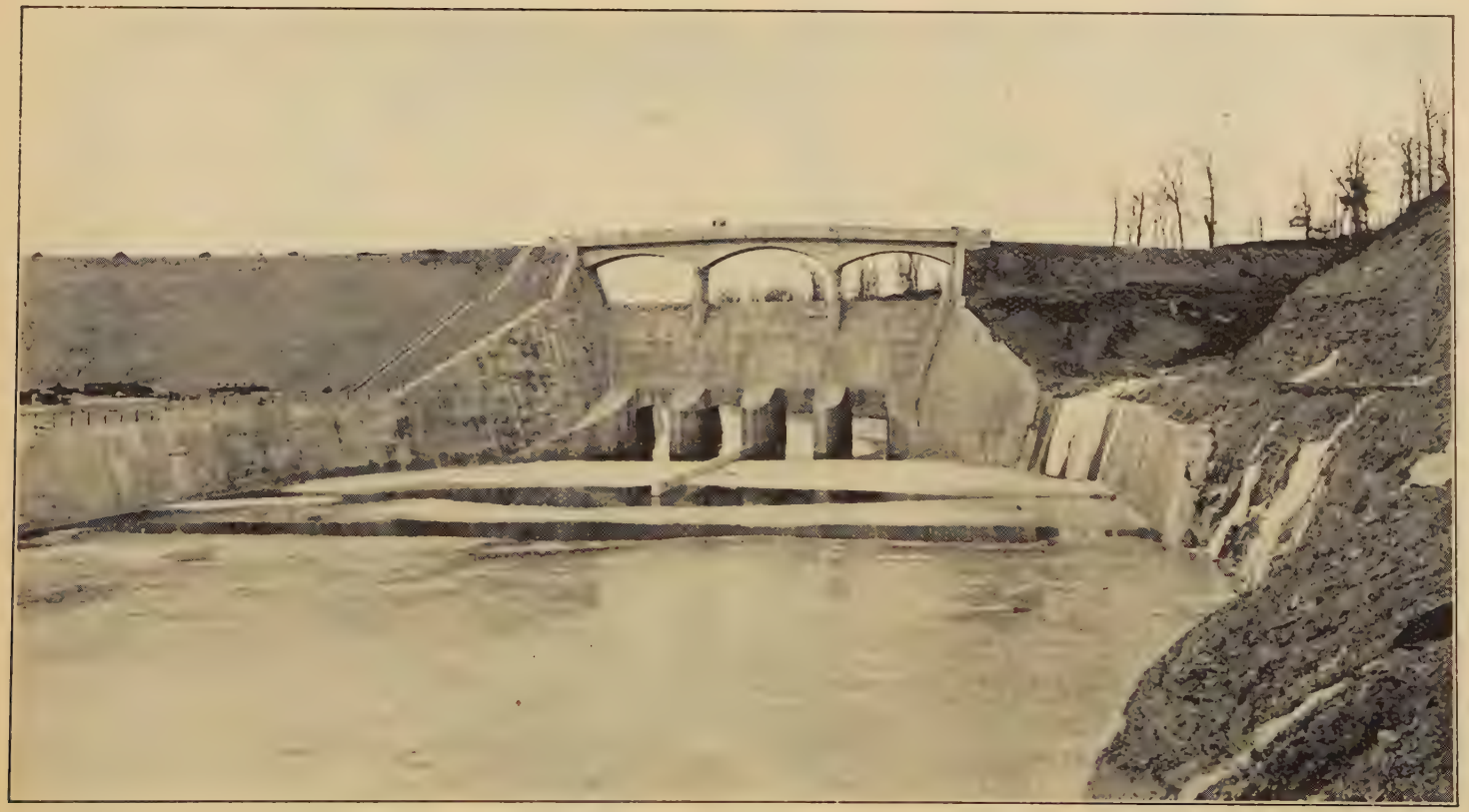

Fig. 38.—Outlet and Spillway Structure, Taylorsville Dam.

formed on the sloping floor, where the concrete is built of sufficient thickness to withstand the action.*

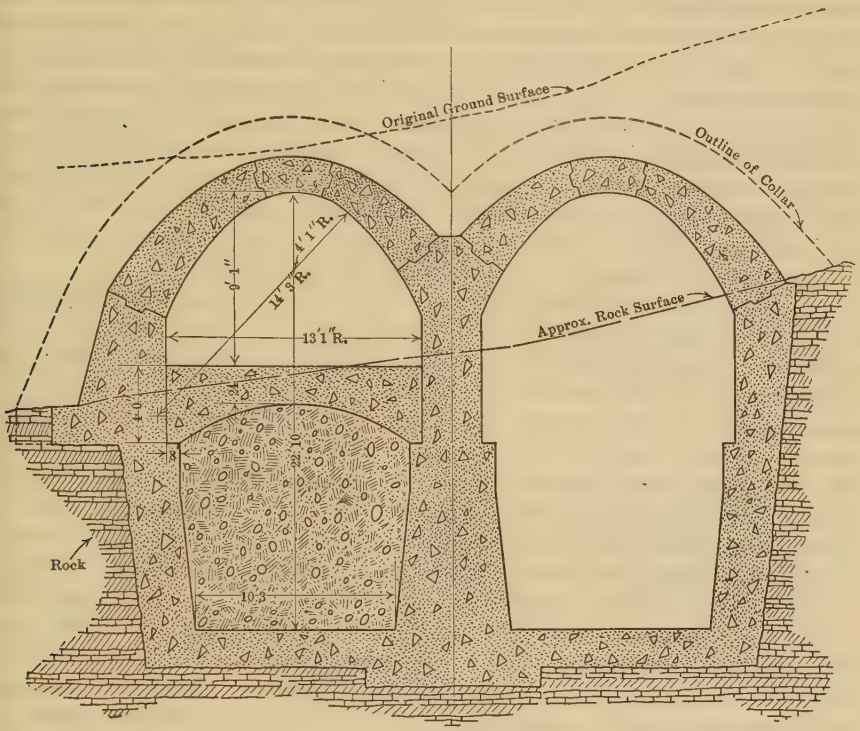

FIG. 39.

Excavation for the outlet structures was performed by dragline machines. A large part of the material had first to be loosened by blasting, care being taken not to disturb the material outside the neat lines. The vertical sides

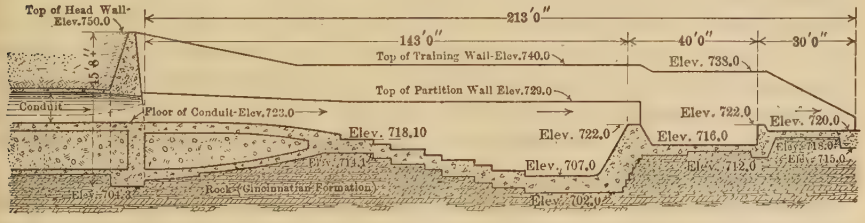

FIG. 40.

of the rock excavation were trimmed by hand to neat lines, and the concrete was placed with no outside forms below the rock surface.

* The experiments leading up to the adoption of this design, and the theory of the hydraulic jump, are discussed in detail in Vol. III of the Technical Reports published by the District, "Theory of the Hydraulic Jump and the Backwater Curves".

At each of the dams a gravel washing and screening plant was erected for furnishing material for the outlet and spillway structures. These plants were standardized and all parts were interchangeable. Suitable gravel and sand for concrete was found near each site. The material was hauled to the plant in dump cars and dumped into a hopper at track level, from which a belt conveyor took it to the top of the plant to the screens. Wash water was delivered to the screens by a small centrifugal pump. The material was washed and separated into three sizes, sand, fine gravel, ranging in size from ¼ in. to 1½ in., and coarse gravel graded from 1½ to 3 in. in size. Each of these sizes passed by gravity into separate storage bins. Over-size material could be either wasted or diverted to the coarse gravel bin. In the heavy walls, it was the practice to use over-size up to about 6-in. cobbles.

A 1-yd. mixer was placed in such position that the material from each of the three bins could be fed to the charging hopper by gravity, through adjustable measuring boxes. Cement was stored in a shed near-by, and was brought to the mixing platform on small warehouse trucks as needed. The mixer discharged into side-dump cars, or bottom-dump buckets on trucks, which were hauled to the work by 3-ton narrow-gauge, gasoline locomotives. At Germantown and Englewood, where the long conduits were built, most of the concrete was chuted into place by gravity. At the other dams, and at the spillway structures at Germantown and Englewood, the concrete was placed by derricks. Collapsible sectional forms were used for the conduit construction at Englewood and Germantown. In building the large retaining walls at the other structures, movable panel forms were generally used. The concrete mix varied from about 1:1¾:3½ in conduit linings, to about 1:3:6 in the bodies of the heavy retaining walls, the change in the mix, within the limits required, being accomplished by changing the number of sacks of cement per batch. Special care was taken to use no more than the required quantity of water per batch. The two sizes of gravel were re-mixed in the batch in such proportion as to use up practically all of each size, and this ratio between fine and coarse gravel was fixed to suit the conditions in the borrow-pit. In other words, no attempt was made to establish a theoretical combination of the fine and coarse gravel, which would have meant a considerable waste of one size or the other from time to time. There was an advantage, however, in having some fine and some coarse gravel in each batch, as this guaranteed more uniform batches than can be obtained from one bin containing all grades of gravel, and was considered sufficient justification for the separation of the gravel into two grades. The sand was carefully proportioned in the batch.

Preparation of Foundations.—The valley floor at each of the dam sites was gravel to indefinite depths, with an overburden—outside the main river channel—of clay varying in thickness from a few inches to several feet, covered with a thin layer of top soil. In preparing the foundations, all top soil and vegetable matter were removed, and roots 1 in. or more in thickness were grubbed out. A cut-off trench, deep enough to serve also as an exploration trench, was dug along the center line of the dam and was afterward filled with core material. As the dams will back up water only in times of

flood, loss of water by underflow is not objectionable, and, therefore, no attempt was made to cut off all the seepage under the dams, but only to force such seepage to travel far enough so that no erosion of embankment material and no possible damage to the structure could occur.

Although sheet-pile cut-offs were used, in most cases, across the old river beds, the principal reliance for control of seepage was placed on blankets of impervious material extending up stream from the core section of the dam. The clay overburden, up stream from the cut-off trench, was examined by means of post-hole diggers and the thin spots were reinforced by artificial "patches" of rolled material. In the old river bed, an artificial blanket of selected material was placed in layers and compacted so that before the hydraulic fill was started, a complete blanket of impervious material, several feet in thickness, covered the dam site up stream from the cut-off trench, connecting with the impervious core of the dam and extending up stream from the up-stream toe as far as necessary to give a ratio, length of seepage travel to head, of about 8 or 9 to 1.

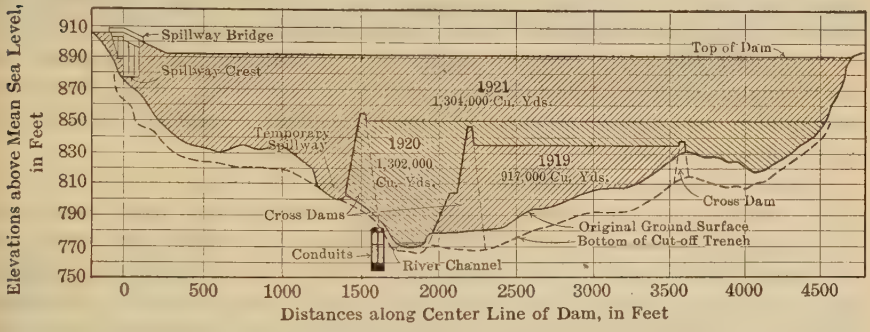

FIG. 41.

River Control During Construction.—At each of the dams, river control during construction was a serious problem, as sufficient waterway had to be maintained to provide for flood flow. The program had to be arranged so that a flood even as large as that of 1913 would not be aggravated by the construction. Although the problem of river control was somewhat different at each dam, the general principles were the same. The procedure at Englewood well illustrates the methods used and Fig. 41 shows the various steps in the construction program. While the conduits were being built on the west bank of the river, a section of the hydraulic fill containing nearly 1 000 000 cu. yd. was constructed on the east side, with a cross-dam along the river bank to hold the semi-fluid hydraulic core in place. During this period, the old river channel remained open, and was left open until the next flood season had passed. Then, another section of fill was placed, extending across the old river channel and over the conduits. Although a season's work at Englewood amounted to 1 000 000 cu. yd. or more, it was impossible to place in one season all the remaining embankment to a height sufficient to turn through the conduits a flood as great as that of 1913. It was necessary, therefore,

during this second season's work on the fill, to leave a temporary spillway at one side of the old river channel. The bottom of this temporary spillway, however, could be left considerably higher than the river bed, and that much was gained toward the final closure. This temporary spillway provided sufficient waterway for protection during the next flood season, and as soon as that had passed, the gap forming that spillway was closed, and it was possible to complete the dam to its full height before the next flood season occurred.

In making these closures of river channels and temporary spillways, certain critical periods had to be anticipated and provided for; for instance, after the river had been diverted, a certain amount of fill could be placed in the closure section before it reached a height at which overtopping would have any appreciable effect on flood flow. From that point, to the time an elevation had been reached sufficient to turn a big flood through the temporary outlet channels, was the most critical period during the construction of the dam, and the safety of the valley below the dams required that this period should be coincident with the low-water season in the river. As a further precaution, it was arranged not to have this stage of construction occur at more than one of the dams at the same time, and every effort was made to rush the work with all possible speed consistent with safety.

Hydraulic Fill.—The hydraulic-fill method of building dams originated in the West and was first suggested by placer mining operations. Where materials are suitable, and proper methods are used, excellent results are obtained at comparatively low cost.

At each of the sites, the embankment material was pumped to the dams by 15-in. dredge pumps, as it was impossible to sluice directly from borrow-pit to embankment. In two cases, however, the borrow-pit material was broken up by hydraulic giants and sluiced to sumps, from which it was pumped into place. At the other three dams, the borrow-pit material was excavated by dragline machines, loaded into 12-cu. yd. dump cars, hauled to the dams by 40-ton standard-gauge, dinkey locomotives, and dumped into shallow bins or "hog boxes." At the "hog boxes," hydraulic giants were used to wash the material to the dredge pumps. Thus, by the time it reached its place in the dam, the borrow-pit material was thoroughly broken up and separated. The discharge pipes from the pumps deposited the material along the outer edges of the dam section, a low levee or dike along the outside slope forcing the discharge to flow toward the center where the core pool was maintained. (See Fig. 42.) The coarser of the material consisting of cobbles, gravel, and coarse sand remained in the outer parts of the embankment, and the fine sand, clay, and silt flowed to the center with the water and settled through the core pool to form the impervious core. Fig. 43 shows another view of one of the dams under construction.

The fill was built in layers, varying in thickness from 2 to 4 ft. at the different dams. Each lift was staked out in advance, and efforts were made to keep the shore line of the core pool straight and even as each layer was carried forward. (See Fig. 42.) Different methods were used for keeping the outside edge of the slope high enough to turn the flow from the dredge pipe toward the central pool. At one dam, this was done by hand; at another,

Fig. 42.—Method of Placing Hydraulic Fill.

Fig. 43.—Taylorsville Dam Under Construction. Low Section of Dam is River Closure Section.

FIG. 44.—GERMANTOWN DAM, COMPLETED, SHOWING HYDRAULIC JUMP IN ACTION DURING MODERATE FLOOD.

teams were used. At the other dams, where 3 or 4-ft. lifts were common, a small dragline machine was used to build up the levees ahead of the fill. Near the end of the work, at one of the dams, this was done by a small back-filling machine with caterpillar traction. This was the most effective of any of the methods used.

The control of the hydraulic-fill cores, both as to gradation of material and rate of consolidation was given careful attention. During construction, the composition of the core may be controlled within certain limits by the selection or choice of borrow-pit material, by wasting the finest of the material from the core, in case a surplus of fines is present, or by securing additional clay from auxiliary borrow-pits, where the main borrow-pits are deficient in fines. There are various methods of controlling core width and of studying the character of core material and rates of consolidation. A description of the core studies at the Conservancy dams has been presented to the Society by the speaker.*

Several kinds of dredge pumps were tested during the early stages of the work. Those that gave the best results had shells of either white iron or manganese steel, with manganese runners and removable manganese shoes on the runner blades. These removable shoes not only doubled or trebled the life of the runner, but also permitted an easy change in effective length of blades, to meet the requirements of increased head as the embankments were raised in height. Special electric-welded dredge pipe of high carbon steel was used with satisfactory results. In service tests, the life of this pipe exceeded that of the old style standard dredge pipe, by 200 to 300 per cent.

Progress and Results.—The rate of progress of hydraulic fill at some of the dams is thought to be worthy of mention. At Germantown, with one dragline machine in the borrow-pit and one dredge pump, an average month's work was about 60 000 to 70 000 cu. yd. During one month the output reached 91 500 cu. yd. At Huffman, with about the same equipment, the rate of progress was about the same. At Taylorsville, with four giants in the borrow-pits and two dredge pumps, the highest monthly estimate reached 107 000 cu. yd. At Englewood, with three dragline machines in the borrow-pit and two dredge pumps, the monthly record of hydraulic fill often reached 150 000 cu. yd., and one month it was 180 000 cu. yd. In the fall of 1917, a progress schedule was set that provided for completion of the five dams at what was believed to be the earliest date consistent with economy. In spite of the necessity for organization of forces and purchase of equipment for construction by the District, delays due to war conditions, priority regulations in employment, purchases, and shipment, car shortage, and generally unfavorable conditions, all the dams were completed, ready to handle a flood, a year ahead of the original schedule.

Although the work is now in such shape that the Valley is protected against a flood like that of 1913, there is still much work to be done, especially on some of the larger of the channel improvement works. The construction will be

* See p. 1181.

completed during 1922. The principal quantities involved in the whole work are as follows:

Public Service Relocations:		
Earth and rock	2 500 000	cu. yd.
Concrete	30 000	" "
Railroad lines	55	miles
Flood-Control Works:		
Earth and rock	19 000 000	cu. yd.
Concrete	250 000	" "

No large floods have occurred since the completion of any of the dams, but two of them have already been tested to the extent of holding back water in the basins to depths of 30 ft. or more. Fig. 44 shows one of the completed dams, with the hydraulic jump in action during a moderate flood.

DISCUSSION

MORRIS KNOWLES,* M. AM. SOC. C. E.—The flood-control problem at Pittsburgh, Pa., which the speaker will discuss, is concerned with four States, as far as inaugurating the policy relates to legal or legislative entanglements. Works will be constructed in Pennsylvania, New York, Maryland, and West Virginia, and beneficial results will be felt in at least two other States and down the Ohio River to Cairo. Thus, the ultimate consummation of the plan is exceedingly difficult.

The comprehensive report of the Pittsburgh Flood Commission revealed the necessity and the extent of the problem, the involved relationship of the many factors, and the need of the correlation of many jurisdictions. Education is necessary, and the project has progressed sufficiently to show the requirements and the methods and machinery which may be established to carry on this work. The Pittsburgh engineers, civic groups, and municipal authorities realize the extent and breadth of the problem, and the danger of an inadequate solution.

Moreover, much has been accomplished; there has been a review by the United States engineers and a recommendation that surveys and plans be made by the War Department, for the purpose of securing flood protection and prevention in certain rivers, and those tributary at Pittsburgh are among such streams. The State of Pennsylvania, in 1919, appropriated $25 000 as its contribution for such surveys and plans, with the idea and request that the United States Government should direct the work through its Army engineers and should appropriate money for this purpose. A bill, known as H. R. 5 357, 67th Congress, First Session, provides for $50 000, to conduct this work and to co-operate with the State of Pennsylvania, is before that Congress.

Great areas at the head-waters of the Allegheny and Monongahela Rivers and their tributaries have already been acquired by the United States for forest reservations, and $50 000 has been expended for this purpose, on the Allegheny water-shed, in Northwestern Pennsylvania. The latter has been made possible by the passage, by the State of Pennsylvania, of an Act enabling the United States Government to acquire such property within the State. The National Forest Commission has recommended the purchase of 500 000 acres in Northwestern Pennsylvania, and about 1 000 000 acres have been secured at the head-waters of the Youghiogheny and Cheat Rivers in Maryland and West Virginia. These areas include vast denuded lands on the upper water-sheds, where the hardwood industry was once active.

The improvement of the wharves at Pittsburgh, including the reclamation of many acres of unused river-front, the erection of dockage facilities, and the construction of flood walls to protect low-lying areas, is now receiving attention through the efforts of Mayor William A. Magee, the United States Engineer's office at Pittsburgh, the Pittsburgh Flood Commission, the Planning Commission, and the Committees of the Metropolitan District of Pitts-

* Director, Dept. of San. Eng., Univ. of Pittsburgh; Cons. Engr., Pittsburgh, Pa.

burgh, all acting as a unit. It is expected that, within a short time, this adequate development of the much abused river-front will be secured and, at the same time, furnish protection from ordinary floods.

The speaker wishes to call attention to the fact that this entire movement and program is only one part of the great community planning idea, namely, a development of the idea of the wisdom of utilizing the great resources of flowing and falling waters to secure the greatest good and benefits to all the people and to all activities. The Pittsburgh Flood Commission was the first to suggest (when this view was not as popular as it has since become, with a growing realization of the difficulties and breadth of the problem) that flood prevention and flood protection be combined and, with wise provision for each, many correlated questions will be solved.

There was the possibility at one time to plan for wise occupation and complete development in the river valleys, but this opportunity has been frequently misused, sometimes by a railroad, a canal, an improved river for navigation, or the location of a town or an industry, but too frequently, it has been the domination of one development and its apparent best interests, without thought of the relationship to others whereby everything could be wisely adapted to promote the greatest development of all activities. It has been realized too late that this is not adequate planning, and communities have frequently been inundated—much to their discomfort and disaster. Engineers now find the problem increasingly difficult, which is true in all cases of replanning.

However, opportunity still exists, as with the proposed wall at Pittsburgh, or the elevation of streets out of ordinary flood levels, to plan for thoroughfares, parking, boulevards, and terminal facilities. Witness the attractive river-front development at Dayton, Ohio, with walls and levees, at Harrisburg, Pa., by the use of concrete walls and parking, or the convenient and pleasing boulevard drive, along the Susquehanna River at Sunbury, Pa., placed on top of a river levee. Thus, through traffic need not be diverted about and around the town. Thus, too, in several river valleys not yet fully developed, the chance still exists of choosing the wisest development. This is true community or country planning.

In true and wise planning, it may be thought best to locate a railroad in one place, an improved channel in another, a reservoir on a wild land in another, forestation on another, etc., so as to obtain the most complete use and activity of land, water, and all related potentialities. The Pittsburgh Flood Commission, the City of Pittsburgh, the County of Allegheny, and the State of Pennsylvania, all unite, therefore, in agreement that there is no general, common, or single panacea to suggest, but that a complete program should be studied and developed.

In harmony with this view, these interests believe that both land and waters—great facilities each—are to be developed to the greatest use. They believe that stored water may have many potential uses, and when released at proper times, under control, between well designed banks and regulated channels, affords a great blessing for the frequently denuded and inundated valleys, holding the water back at times of danger, letting it out at dryer times,

to the improvement of navigation, development of power, increase of water supply for various purposes, dilution and oxidation of waste products, and the general betterment of all unpleasant conditions. Furthermore, that a single solution which forgets, hinders, or hampers other developments, is unwise. It is hoped and expected, that the efforts in education and propaganda will result in that wiser development which will regard all factors and all uses and result in what is called complete community planning.

HARRISON P. EDDY,* M. AM. SOC. C. E.—One of the most important questions to be solved in any flood problem is that of the maximum flood flow for which provision must be made in the works to be designed. It may often be impracticable to provide protection against the greatest flood which must be expected at rare intervals, but, nevertheless, the greatest probable flood must be considered and the works must be made safe against serious damage under the most trying conditions, even though they do not fully protect the community against loss at such times. It may be necessary to provide spillway or waste-channel capacity for the greatest probable flood, since, in the majority of cases, failure of the dam due to inadequacy of the spillway might result in more serious disaster than would be probable from an inadequacy of stream channel.

A great many dams have been built without adequate spillway capacity; many of them were built without adequate engineering design; others, designed when comparatively little information on extreme floods was available, were given what was supposed to be adequate spillway capacity, but much less than would be provided at present. The paucity of information relating to flood flows and necessary spillway capacities in nearly all the treatises on dams, is startling. In fact, many of them make no references to the subject. The old rule, that a spillway should be capable of passing a 6-in. depth of water on the drainage area in 24 hours, has long been known to be unscientific, and, in many cases, has resulted in inadequate provision, yet it is still quoted and apparently with approval in some recent reference books. The subject has been fully discussed by F. W. Scheidenhelm, M. Am. Soc. C. E.,† and his suggestions, if generally adopted, would lead to a radical departure in the liberality of spillway design from that more or less commonly adopted in the past.

Much more liberal provision for discharging storm water is certain to be required in the future. Many existing dams will have to be modified to provide adequate spillway capacity, and some of this work is being done, noticeably in Pennsylvania, where the State Water Supply Commission is investigating the adequacy of spillways and requiring owners to make changes where necessary.

The data on which to base a decision of the maximum flood to be expected in any given locality, are comparatively few and must be supplemented by a detailed study of rainfall and run-off records. Such data should also be checked carefully against records of excessive floods throughout the United States. In view of the difference in climatic conditions, it is somewhat sur-

* Cons. Engr. (Metcalf and Eddy), Boston, Mass.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 907.

prising that the relation between maximum flood discharge and extent of drainage area appears to be approximately the same through the United States, although there may be a decided difference in the frequency with which such floods occur. In spillway design, the maximum discharge, rather than the frequency, would probably be the governing factor.

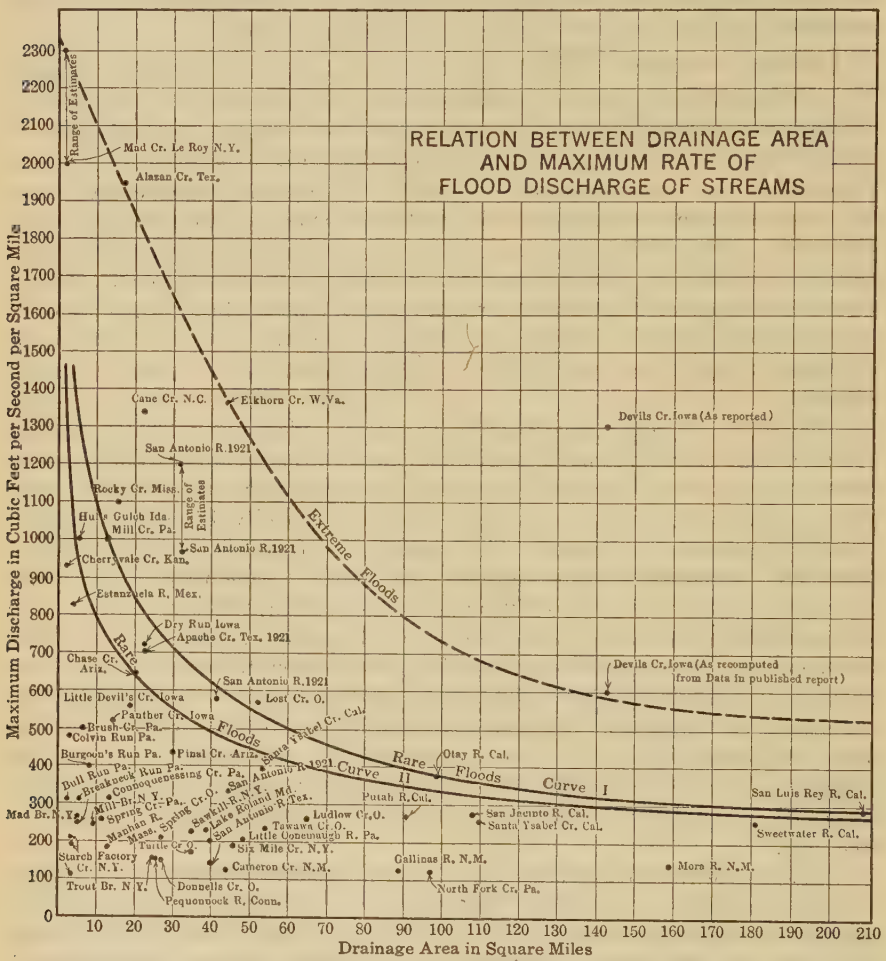

FIG. 45.

Records of floods of unusual magnitude have been plotted on Fig. 45, on which three curves are shown. The upper curve represents the maximum floods of which apparently authentic records are available—floods which are probably to be classed as "acts of God". In addition to the four floods shown on this diagram, support is also given to this curve by one from a much larger area—the flood of 1909 in the Santa Catarina River at Monterrey, Mexico.*

* At least three other flood records supporting the curve of maximum floods, which were not available to the speaker when the diagram was prepared, are given in Mr. Matthes' Table 1 (p. 1393), as well as several records of floods which would support the curves of rare floods or lie between those and the curve of extreme floods.

These flows are so great that it will probably be impracticable to make provision for protection against flooding resulting from them, but their possibility should be borne in mind in guarding against failure of structures the collapse of which might accentuate the damage. Curve I and Curve II, Fig. 45, both intended to represent floods of rare occurrence, indicate the magnitude of the flood flows for which provision should in general be made in structures intended to minimize flood damages.

Table 5 gives the data from which the diagram was constructed, and the sources of the information.

It is not unlikely that, as Allen Hazen, M. Am. Soc. C. E., has stated*: "It may be expected that the continuous records of gauging stations will ultimately be more useful in establishing these laws than records of destructive floods that occur now and then where there is no continuous gauging." Until much more is known, however, about the relations between frequency and magnitude, it will still be necessary to rely largely on the scattered records of extreme floods, especially for spillway design.

The best method yet proposed for predicating maximum flood flows on data obtained from continuous gaugings is doubtless that of Weston E. Fuller, M. Am. Soc. C. E.† An attempt to apply Mr. Fuller's method to a stream for which the record of gaugings does not extend over a considerable period of years, is likely to be unsatisfactory; at least, that was the writer's experience in studying the flood problem of San Antonio, Tex. In the report submitted by his firm to the City of San Antonio on December 6th, 1920, is a discussion of the application of the Fuller formula to the San Antonio River, in which it was shown that, using the data on "Western Gulf of Mexico Streams", quoted in Mr. Fuller's paper previously mentioned, the largest coefficient indicated for that region is 31, which would correspond to flood-flow rates of 64 and 84 sec-ft. per sq. mile for frequencies of 100 and 1 000 years, respectively, and a drainage area of 41 sq. miles. Taking such records of the actual flow of the San Antonio River as were available in 1920, but using maximum rates of flow for the largest floods instead of maximum daily average flows, the coefficient of the San Antonio River was estimated at 167, five and one-half times as large as was indicated by Mr. Fuller from the data available at the time his paper was prepared. Yet the application of the formula with this coefficient indicates flood flows of only 345 and 450 sec-ft. per sq. mile, for frequencies of 100 and 1 000 years, respectively. The actual rate of flow in the 1921 flood, according to an estimate made by C. Terrell Bartlett, M. Am. Soc. C. E., was 580 sec-ft. per sq. mile‡

A most extraordinary flood discharge from a small drainage area is reported in the Water Resources Inventory Report§ of the Pennsylvania Water Supply Commission, as follows: "The highest recorded flood occurred on Bull's Run, a small tributary of the Susquehanna River‖ on July 15th, 1914, which discharged at the rate of 5 000 sec-ft. per sq. mile from a drainage area

* See p. 367.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 607.

‡ See p. 362.

§ "Floods", Part VIII (1917), p. 18. Also, pp. 70 and 128 of Vol. III, "Gazetteer of Streams", where the drainage area is given as 0.7 sq. mile.

‖ South of Wrightsville, Pa.

TABLE 5.—Flood Flow of Streams.

Stream and locality.	Drainage area, in square miles.	Flood flow, in cubic feet per second per square mile.	Date of flood.	References.
Mad Creek, Le Roy, N. Y.	1½	2 000 to 2 300	May, 1916	*Eng. Record*, Vol. 73 (1916) p. 842.
Cherryvale Creek, Cherryvale, Kans.....	2	930	(?)	*Eng. News-Record*, July 3, 1919, p. 31; *Transactions*, Am. Soc. C. E., Vol. LIV, p. 200, and Vol. LXXVII, p. 658.
Bull Run, Jeannette, Pa.....	2.25	310+	July 5, 1903	Pa. Water Supply Comm. Rept., 1912, p. 50.
Colvin Run, Grindstone, Pa...	2.7	480	July 21, 1912	Pa. Water Supply Comm. Rept., 1914, p. 45.
Starch Factory Creek, New Hartford, N.Y	3.4	108 109 190 209	March, 1903 March 25, 1904 June 21, 1905 Sept. 3-4, 1905	*Water Supply Paper 147*, p. 37; *Water Supply Paper 162*, p. 12; N. Y. State Engr. Rept., 1905, p. 690.
Estanzuela River, Monterrey, Nueva Leon, Mexico.....	3.5	825	Aug. 27-28, 1909	*Eng. News*, Sept. 23, 1909, p. 315.
Hulls Gulch, Boise, Idaho.....	5.	1 000	July 24, 1913	*Monthly Weather Review*, July, 1913.
Mad Brook, Sherburne, N. Y..	5.0	262	Sept 3-4, 1905	*Water Supply Paper 162*, p. 12.
Breakbeck Run, Bullskin Township, Pa.....	5.2	310 250	May 19, 1902 Aug. 19, 1912	Pa Water Supply Comm. Rept., 1914, p. 47.
Brush Creek, Jeannette, Pa...	6.	500+	July 5, 1903	Pa. Water Supply Comm. Rept., 1912, p. 69.
Burgoon's Run, near Altoona, Pa.....	8.1	400+	May 20, 1894	Pa. Water Supply Comm. Rept., 1912, p. 59.
Mill Brook, Edmeston, N. Y...	9.4	241	Sept. 3-4, 1905	*Water Supply Paper 162*, p. 12.
Spring Creek, Harrisburg, Pa.	11.6	238	Feb. 15, 1908	Pa. Water Supply Comm. Rept., 1910-11, p. 930.
Mill Creek, Erie, Pa.....	12.9	1 000	Aug. 3, 1915	*Eng. News*, Nov. 11, 1915, p. 937.
Manhan River, Holyoke, Mass.	13.0	182	Feb. 13, 1900	Letter of James L. Tighe, M. Am. Soc. C. E., June 13, 1919.
Connoquenessing Creek, Oakland, Pa.....	13.6	315	Aug. 28, 1903	Pa. Water Supply Comm. Rept., 1914, p. 48.
Panther Creek, Iowa.....	14.	520	June 10, 1905	*Water Supply Paper 162*, p. 26, and Meyer's "Hydrology", p. 134
Rocky Creek, near Ellisville, Miss.....	15.	1 110	May 7, 1882	*Transactions*, Am. Soc. C. E., Vol. XXV, p. 116.
Alazan Creek, San Antonio, Tex.....	16.9	1 950	Sept. 9, 1921	See pp. 365 and 368.
Little Devil's Creek, Iowa.....	19.	560	June 10, 1905	*Water Supply Paper 162*, p. 26, and Meyer's "Hydrology", p. 134.
Chase Creek (tributary of Gila River), Arizona.....	20.	647	Dec 1906	*Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 583.
Cane Creek, Bakersville, N. C.	22.	1 341	May 20, 1901	*Eng. News*, Aug. 7, 1902, p. 103.
Apache Creek, San Antonio, Tex.....	22.	704	Sept. 9, 1921	See pp. 365 and 368.
Dry Run, Decorah, Iowa.....	22.3	720	March 15, 1919	*Eng. News-Record*, March 18, 1920.
Trout Brook, Brooksport, N.Y.	25.	158	.....	*Transactions*. Am. Soc. C. E., Vol. LXXVII, pp. 572 and 614.
Pequonnock River, Bridgeport, Conn.....	25.	157	July 29-30, 1905	*Water Supply Paper 162*, p. 1.
Spring Creek, above Dayton, Ohio.....	27.	210	March 23-27, 1913	Miami Conservancy Dist., Technical Repts., Part IV, p. 60.
Donnels Creek, above Dayton, Ohio.....	27.	147	March 23-27, 1913	Miami Conservancy Dist., Technical Repts., Part IV, p. 60.
Pinal Creek, Globe, Ariz.....	30.	440	Aug. 17, 1904	*Eng. News-Record*, July 3, 1919, p. 31; *Water Supply Paper 147*, p. 118.
Sawkill River, Kingston, N.Y.	35.	228	Apr. 1895 also 1896	*Water Supply Paper 35*, p. 61.
Turtle Creek, above Dayton, Ohio.....	35.	175	March 23-27, 1913	Miami Conservancy Dist., Technical Repts., Part IV, p. 60.
Lake Roland, Md.....	39.	230	1868	*Transactions*. Am. Soc. C. E., Vol. LXXVII, pp. 572 and 614.
San Antonio River, San Antonio, Tex.....	40.	140 200	Oct. 23, 1914 Dec. 2, 1913	*Water Supply Paper 438*, p. 59.
San Antonio River, San Antonio, Tex.....	32.4 41. 45.	960 to 1200 580 333	Sept. 9, 1921	See p. 362.

TABLE 5.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Flood flow, in cubic feet per second per square mile.	Date of flood.	References.
Cameron Creek, Hurley, N. Mex.	44.	125	Aug. 14, 1913	*Water Supply Paper 358*, p. 662.
Elkhorn Creek, Keystone, W. Va	44.	1 363	June 22, 1901	*Eng. News*, Aug. 7. 1902, p. 104.
Sixmile Creek, Ithaca, N. Y.	46.	195	June 21, 1905	*Water Supply Paper 162*, p. 3.
Pine Creek, Paris, Tex.	48.	320 to 410	May 12, 1920	Letter from John B. Hawley, M. Am. Soc. C. E.
Little Conemaugh River, Johnstown, Pa.	48.6	206	May 30-31, 1889	*Transactions*, Am. Soc. C. E., Vol. XXIV, p. 431.
Lost Creek, above Dayton, Ohio	52.	571	Mar. 23-27, 1913	Miami Conservancy Dist., Technical Repts., Part IV, p. 60.
Santa Ysabel Creek, near Mesa Grande, Calif.	53.4	395	Jan. 27, 1916	*Water Supply Paper 426*, p. 56.
Tawawa Creek, above Dayton, Ohio	54.	239	Mar. 23-27, 1913	Miami Conservancy Dist., Technical Repts., Part IV, p. 60.
Ludlow Creek, above Dayton, Ohio	65.	266	Mar. 23-27, 1913	Miami Conservancy Dist., Technical Repts., Part IV, p. 60.
Dry Creek, Pueblo, Colo.	70.	300	June 2, 1921	See p. 48.
Gallinas River, Las Vegas, N. Mex.	89.	131	Sept. 29, 1904	*Water Supply Paper 147*, p. 138.
Putah River, Guenoc, Calif.	91.	270	Mar. 10, 1904	*Eng. News-Record*, July 3, 1919, p. 31; *Water Supply Paper 298*, p. 370.
North Fork Creek, Brookville, Pa.	97.	124	July 17, 1912	Pa. Water Supply Comm. Rept., 1912, p. 49.
Otay River, Lower Otay Dam, Calif.	98.6*	379	Jan. 27, 1916	*Eng. News-Record*, July 3, 1919, p. 31; *Water Supply Paper 426*, p. 44.
San Jacinto River, San Jacinto, Calif.	108.	278	Jan., 1916	*Eng. News-Record*, July 3, 1919, p. 31; *Water Supply Paper 426*, p. 70.
Santa Ysabel Creek, Ramona, Calif.	110.	258	Jan. 27, 1916	*Eng. News-Record*, July 3, 1919, p. 31; *Water Supply Paper 426*, p. 56.
Devil's Creek, Iowa	143.	600±	June 10, 1905	*Water Supply Paper 162*, p. 26, Meyer's "Hydrology", p. 134.
Mora River, La Cueva, N. Mex.	159.	140	Sept. 29, 1904	*Water Supply Paper 147*, p. 127.
Sweetwater River, Sweetwater Dam, Calif.	181.	251	Jan. 27, 1916	*Eng. News-Record*, July 3, 1919, p. 28; *Water Supply Paper 426*, p. 48.
Cave Creek, Phoenix, Ariz.	200.	125	Aug. 21, 1921	*Eng. News-Record*, Sept. 15, 1921, p. 464.
San Luis Rey River, Mesa Grande, Calif.	209.	280	Jan. 18, 1916	*Eng. News-Record*, July 3, 1919, p. 31; *Water Supply Paper 426*, p. 61.
San Luis Rey River, Oceanside, Calif.	561.	125	Jan. 26, 1916	*Eng. News*, Feb. 24, 1916, p. 383.
Santa Catarina River, Monterrey, Nueva Leon, Mexico.	544.	432±	Aug. 27-28, 1909	*Eng. News-Record*, July 3, 1919, p. 28; *Eng. News*, Sept. 23, 1909; *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 523.
Fountain Creek, Pueblo, Colo.	930.	54	June 3, 1921	See p. 58.
Arkansas River, Pueblo, Colo.	1 740.	57	June 3, 1921	See p. 10.

* Measured on topographic map; 12.7 sq. miles tributary to Upper Otay Reservoir.

a little over ½ sq. mile." This rate of run-off is equivalent to 7.8 in. of water per hour on the drainage area. This would be an extremely high rate of precipitation for a period as short as 5 min.; while the period of concentration for a drainage area of 450 acres would probably be not less than 20 min. It may be, therefore, that this phenomenal flow resulted from the giving way of some obstruction rather than from the natural run-off following an excessive rain, and this record has accordingly been omitted from Table 5. Rates of precipitation as high as 7.80 in. per hour, for periods of 20 min., have been reported by the U. S. Weather Bureau only twice during the 18

years, 1901–18, inclusive,* as follows: April 29th, 1905, Taylor, Tex., 8.25 in. per hour, and, October 20th, 1909, Pensacola, Fla., 8.64 in. per hour.

J. Albert Holmes,† M. Am. Soc. C. E.—In the structures described by Charles H. Paul, M. Am. Soc. C. E., composed of impermeable cores and permeable dikes, the core is the real "dam", but its entire success depends on the stability and rigidity of the dikes.

In geological formations, clays and sands, formed by ice or streams, are deposited in water and, as great depths are laid down, are subjected to enormous pressure, converting the clays to shales and slates and the sands to sandstones and quartzites. Similar action takes place to a less extent in earth-fill dams and to determine its extent and effect, some experiments have been made on the core materials of two dams,‡ in which samples of the core were placed in a metal tank, pressure applied to the material at increasing rates, and the quantity of seepage measured after each pressure application. In these experiments, it was found that when the material was first placed in the test tank and before settlement had taken place or pressure had been applied, the seepage rate was high.

This loose condition and high seepage rate is analogous to the conditions existing in the upper layer of a core when first deposited in the pool. As the overburden increases, the core is consolidated, the voids are decreased, the surplus water is expelled, and the porosity is diminished. In the experiments, an increase in pressure from 150 to 3 400 lb., about 1½ tons per sq. ft., caused a reduction in the seepage rate of more than 90% and a decrease of 22.7% in volume of material from the loose condition.

The results of the tests, plotted on logarithmic paper, give the data in Table 6, in which the maximum rate or quantity of water that will filter through the core material of the two dams, with a loss of head equal to the thickness of the sample, is given in gallons per acre per day.

Within the limits of the test, the compression amounted to 20%, with an application of only 1 ton per sq. ft.; under additional pressure, the consolidation was less rapid, 2 tons per sq. ft. increasing it only 4 per cent. The percentages of settlement are based on dry, loose material and are to be considered as only approximate beyond the limits of the pressures applied. The seepage quantities are reduced to a uniform temperature of 50° Fahr.

Darcy's law of the flow of water through a column of soil, in which the flow is directly proportional to the head and inversely to the length of the column, is modified by the effective size of the soil grain, the temperature (viscosity of the water), the percentage of voids, the porosity, and the presence of vegetable and colloidal matter.

The effective size of the soil grains in dam cores is small. They are so small, in fact, that the flow does not seem to be influenced by the size to nearly the same extent as in filter sands.

* *Engineering News-Record*, Vol. 82 (1919), p. 1066.

† Boston, Mass.

‡ "Some Investigations and Studies in Hydraulic-Fill Dam Construction", by J. Albert Holmes, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 331–358.

TABLE 6.

Pressure, in pounds per square foot.	Percentage of compression, approximate.	Seepage at Dam No. 3, in gallons per acre per day, 1:1 slope.	Seepage at Dam No. 4, in gallons per acre per day, 1:1 slope.
2 000	20	20 500	63 500
4 000	24	11 300	41 700
6 000	27	10 700	35 300
8 000	29	9 000	29 500
10 000	31	7 900	25 600
12 000	33	7 100	22 900
14 000	35	6 500	20 600
16 000	37	6 000	19 000

In experimenting with small volumes of material, the temperature (viscosity of the water) is important. To determine its actual temperature within the sample is difficult, but in order to get correct results this should be done. To obtain temperatures in the core of a dam would be still more difficult, but it could be accomplished, although the probabilities are that the slow moving water does not change much within the great mass of earth, but retains a nearly uniform temperature, as does the water issuing from springs.

Percentage of voids (porosity) is of great importance. To their reduction by pressure (compacting) is due the stability and imperviousness of the core; imperviousness is increased by the presence of vegetable and colloidal matter.

It has been shown that dry clay, when confined and not allowed to expand, will absorb only about 3% of water, but if permitted to expand, it will take up water in proportion to its increase in volume. Also, that clay under pressure will give up its water in the presence of water.

Equal pressures reduce like materials to the same consistency, that is, a pressure of 3 tons per sq. ft. will reduce the moisture content of a pure clay to about 27%, and repeated trials bring about the same results without regard to the quantity of water originally present. A moisture content of 27% corresponds to about 50% of voids and, in the experiments, the clay was reported as "much hardened." An increase of pressure to 6 tons per sq. ft. reduced the moisture to 23% and the voids to 44 per cent.

These results come about quickly under experimental conditions using small volumes and more gradually in the large masses of material in the cores of the dams.

With stable and immovable dikes, compactness and stability of the core are sure to exist, and the speaker's thought is that, perhaps, where this has not occurred and movement of the dikes was taking place at intervals, the core retained its moisture and volume, although, later, when the structure had changed its form and had become stable, the same material consolidated and gave up its moisture content.

D. W. MEAD,* M. AM. SOC. C. E.—In 1914, the speaker had an opportunity to see some of the country described by John R. Freeman, President, Am. Soc. C. E., in his discussion of flood problems in China, and was also impressed

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with the ingenuity of the Chinese in their utilization of local resources for flood protection, such as clay and reeds—the only materials available in many places along the Grand Canal—to prevent erosion and to build temporary dams.

The speaker, however, was most impressed by the fact that here was a people who had 4 000 years of practical experience in hydraulic work, and although successful in minor details, they had, on the broad general principles, made an utter failure of their projects, because they had no correct fundamental theory on which to base their work. Those things that they could learn by practical experience, had been learned well—masonry work, protection work, and many things in which experience really counted; but on broad principles they had had no advantage of a sound theoretical basis, and their work, in general, was exceedingly defective.

Chinese education has been classical, not scientific, and their great men, educated in the Chinese classics and with no knowledge of science, have been unable to correlate the experience of the past and apply it to the practical uses of the present. They recognize an outlet as necessary for the discharge of flood water, but have no conception of capacity. One of their most highly educated men conceived, for the Huai River floods, an outlet 200 ft. wide by 10 ft. deep, which should have been 2 000 ft. wide by 20 ft. deep. To the solution of these great flood problems, China has brought centuries of experience, but no scientific knowledge, and years of practice without a correct theory. The impression left on the speaker's mind, by his observations in China, was the necessity of sound theory as a basis for correct practice.

The school of practical experience is exceedingly valuable, and usually correct theoretical analysis must be demonstrated by the results of such practice before the theory is of much value or can be safely adopted. As a basis of future practice, practical experience is, however, of comparatively little value unless the conditions are considered and studied and all the facts correlated and combined into a correct theory, from which experience can be extended to greater things. Without correct theory, the value of practical experience is limited to a reproduction of the experience under identically similar conditions, and as no two sets of conditions are ever exactly similar, the experience so applied is liable to failure on account of the introduction of factors which the previous experience does not include.

Real advancement is attained only through a combination of theory and practice. Each is incomplete and often dangerous without the other, but, by parallel development, safe and substantial progress is possible.

It is gratifying that so many members of the Society are working along these lines of investigating the facts, correlating them, developing the theory that will explain those facts, and carrying out effectively work for protection against floods.

Arthur O. Ridgway,* M. Am. Soc. C. E.—In Colorado and in the Colorado Rockies, all floods are of the cloudburst type. The Continental Divide describes a sinuous course through the State of Colorado in a general northerly

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and southerly direction. It covers 2° of longitude—1° on either side of the 106th Meridian—is about 290 miles in length, and at no place in that length is it less than 10 000 ft. in altitude. Little trouble with cloudbursts is experienced on the western slope of the Rockies. The eastern slope receives much less precipitation than the western slope, especially in the form of snow, and there all the floods come from cloudbursts.

The first problem in the floods of the Colorado Rockies is the distribution of the precipitation. In one drainage area, the mean annual precipitation will vary from 9 to 25 in., computed over a period of 40 or 50 years, and of this, rainfall contributes much the larger proportion. The rainfall does not come in general storms; it is practically all confined to sudden and intense thunderstorms, of which there is often a rapid recurrence. There may be three or four in the same locality during an afternoon. Apparently, there is no law of their occurrence; in one tributary, there may be a severe storm with the next tributary dry.

Another problem is the suddenness and the intensity of the storms. The clouds gather quickly, drop their load as it were, and are gone. The movement of the storms is peculiar. Recently, a series of storms moved down the Colorado River on one side of the drainage basin and, at the same time, another series was moving up the river on the opposite margin.

There are no authentic records as to the intensity. The weather stations, although quite close together, are inadequate for determining precipitation of these intense and highly concentrated storms. In an area over which the precipitation will average 10 to 15 in. per year, as much as 14 in. of rainfall is known to have fallen in a single afternoon or evening, and as much as 5 in. in 30 min. At Pueblo, in the Arkansas drainage, two storms are recorded in which about 2.0 in. fell in less than 1 hour or, to be more exact, one storm of 2.02 in. in 50 min. and another of 1.78 in. in 36 min.

The next problem is coincidence. There are many factors that militate against the determination of flood flows in tributaries. The precipitous slopes, the irregularity of the gradients of the streams, and the recurrence of the storms themselves, with apparently no law of occurrence, make it impossible to compute with any degree of reliability the probable coincidence for the floods in the tributaries.

Perhaps the greatest problem is contributed by the high velocities of flow in the streams of the Colorado Rockies. The country, literally, stands on end; stream velocities of 18 to 20 ft. per sec. are common, which make it difficult to deal with floods in this region. The high velocities bring down a great deal of débris, which helps to give additional power to the water, so that the floods are very destructive.

The suddenness and intensity of the storms with their enormously high run-off rate may be illustrated by the Arkansas River flood of June, 1921. The discharge of the river increased forty or fifty times in 7 hours. Some tributaries of the Arkansas, dry at the beginning of the storm, were, a few minutes later, discharging from 35 000 to 40 000 sec-ft.

Summarizing, the speaker would state that the real flood problems in the Colorado Rockies are: First, erratic distribution of rainfall; second, great

intensity of storms over small areas and their rapid recurrence; third, difficulty of foreseeing or foretelling the coincidence of tributary floods; and, fourth, the high velocity of the flood waters.

There might be added another problem of rather a psychological or sociological character. People want the water and, to them, it seems no less than an economic crime to provide means for its getting away. If the channels are improved, their discharge is increased, and the water runs away more rapidly. Especially on the eastern slope of the Rockies, is it wanted for irrigation. Therefore, when flood protection is attempted, one gets into difficulty because of the fact that every one wishes to conserve all the water.

J. B. CHALLIES,* M. AM. SOC. C. E. (by letter).—The river-control problems that have been solved in Canada have been largely for power purposes, but in many cases these projects incidentally result in bettering flood conditions.

La Loutre Reservoir.—The La Loutre Reservoir on the St. Maurice River, in the Province of Quebec, was constructed and operated by the Quebec Streams Commission. It was completed in 1917 and has a capacity of 160 000 000 000 cu. ft. This storage has increased the power available on the river by 400 000 h. p., and the control is also of much benefit to logging operations, particularly in breaking log jams by controlled flood waves released from the reservoir. The Quebec Streams Commission also carries out important storage operations on the St. François River, mainly for the benefit of the numerous power developments on that stream.

Ottawa River Storage.—Storage undertakings on the Upper Ottawa River, carried out by the Dominion Department of Public Works, include three reservoirs with a total capacity of 125 000 000 000 cu. ft., used to benefit the large power sites on the river, the potential power of which is more than 1 000 000 h. p., and also logging and navigation.

Trent Canal System.—The two examples previously mentioned are typical of control from one or a few large storages, whereas in the Trent River System, the control is from a large number of relatively small storages. Dams have been built at the outlets of many lakes on tributary streams, which supply the canal system on this river and supplement the minimum flow for power purposes.

Although the various storage works mentioned are primarily for power development and navigation, they have a marked effect on the control of floods. Many of the Canadian rivers present problems in which flood control is of primary importance, and a few typical examples in the various Provinces may be of interest.

Floods in British Columbia.—In British Columbia, a disastrous flood occurred in the autumn of 1921. A rainfall of more than 15 in. in 26 hours caused the Coquitlam River to flood, resulting in much destruction. A discharge of 24 000 sec-ft. was recorded, as compared with a previous recorded maximum of about 12 000 sec-ft. A consoling feature was that the dam controlling the waters of Coquitlam Lake withstood this test and that the factors

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of safety insisted on, in its construction, by John R. Freeman, President, Am. Soc. C. E., have been justified. The Coquitlam Dam, the first hydraulic-fill dam built in Canada, is 1 200 ft. long with a maximum height of 100 ft. and an extreme width at the base of 655 ft. On the same occasion, in an adjacent water-shed, at Britannia Mines, even greater calamity was caused by flood, including the loss of thirty-six lives. The power and storage dams did not give away, and the flood was probably lessened greatly by them.

The problems which have to be faced at the delta of the Fraser River in this Province are also associated with flood prevention. The discharge of this river has varied from a maximum of about 380 000 sec-ft. to a minimum of 12 000 sec-ft. and accumulations of snags, trees, and silt must be removed regularly and dredging operations have to be carried on for navigation, thus, also, preventing serious floods. Silt problems in this river resemble somewhat those of the Mississippi, and interesting results are anticipated from the co-operative study undertaken by the Dominion Department of Public Works and other Government and technical organizations as to their best solution.

Floods in Alberta.—In the Province of Alberta, floods often result from high summer temperatures which cause an excessive melting of the snows in the mountains.

In June, 1915, the North Saskatchewan River at Edmonton rose 38 ft. above low water and 800 families were temporarily rendered homeless by this flood. To the south, in the same Province, floods have occurred on the Bow River in 1879, 1884, 1897, and 1902, when, in each instance, a part of the City of Calgary was under water. Escape from flood in 1915 is attributed to the weather turning suddenly cold during the warm period which caused the Edmonton flood, thus stopping the excessive melting of snow in the mountains.

The many uses of the Bow River, including water supply, irrigation, and water power, would tend to encourage further artificial control of its waters, but the low winter temperature, subject to sudden changes, presents difficult conditions for efficient storage operation.

Floods in Manitoba.—In Manitoba, records of floods on the Red River of the North are available from various sources for a great number of years, and floods, reported as particularly serious, occurred in 1776, 1790, 1809, 1826, 1852, and 1861, when, in each case, the plain on which the City of Winnipeg is built was inundated. The river drains to the north, and the spring thaw in its upper water-shed brings down flood waters to the lower reaches before the complete release of ice has occurred and the contracted channel below Winnipeg also contributes to flood conditions. The control methods considered include reservoirs in the upper drainage areas and overflow discharge from the Assiniboine River, a large tributary, to an adjacent basin, together with dikes and improvement of the river channel. Following the flood damages of 1916, in that part of the basin in the United States, the co-operation of the Manitoba and Dominion Governments was requested to remedy conditions.

Floods in Ontario.—Lake of the Woods, between the Provinces of Manitoba and Ontario, has lately furnished a typical example of how flood conditions

can be alleviated materially by proper storage control by a responsible public organization.

In 1916, previous to the placing of the regulation of the Lake under the direction of the Lake of the Woods Control Board, a notable flood occurred in this drainage area. In contrast with this, in 1919, when the Lake of the Woods Control Board was exercising its jurisdiction, conditions which naturally would have had far more disastrous results, were anticipated and coped with successfully. Prompt action at the control works was made possible by telegraphic communication from the various meteorological stations reporting heavy rainfall, and although the inflow rate to the Lake became twice that of 1916, systematic and intelligent regulation enabled the Board to cope successfully with the situation.

Farther east, in Ontario, the Grand River presents a flood-control problem which, during the past decade, has often been discussed at professional meetings. The situation is aggravated by the large number of important centers at various points along this river, which are more or less affected by the undue rise of its waters. Among these centers are Brantford, Kitchener, Woodstock, Galt, Waterloo, Preston, Hespeler, and Paris.

The recent recorded flow of the Grand River shows a maximum of 37 000 sec-ft. and a minimum of only 24 sec-ft., and its floods are attributed to the destruction of great forest areas. The drainage of hundreds of square miles of swamps in the head-waters have changed moderate floods and sustained flow to destructive floods and gradually disappearing flow.

The floods of the Grand River are caused by spring freshets, and storage works for combined flood control, water power, and irrigation have been suggested.

St. Lawrence River.—Trouble from flood would hardly be expected on the St. Lawrence, with its great natural reservoirs and well regulated flow. This stream, however, furnishes an example of floods caused, not by an excessive discharge, but by the blocking of its channel by ice at certain points. The disastrous floods on the St. Lawrence, in the vicinity of Cornwall, in 1840, 1879, 1887, 1895, 1898, 1901, and 1905, were caused by ice jams. In February, 1887, ice jams caused the water to rise 31 ft. above normal summer level, at a point above Cornwall, inundating two-thirds of that town.

Floods in Quebec.—In the Province of Quebec, a disastrous flood occurred on the Chaudière River in the latter part of July, 1917, and serious spring floods have been recorded on this river in 1885, 1896, and 1912. Its most important tributaries, rapidly descending their steep courses, enter the main stream within a distance of 13 miles, just above a long section of the river that has low banks and a narrow channel, where the effect of floods is particularly disastrous. The Chaudière flows to the north, and the spring floods are caused by freshets and ice accumulation from its southern drainage reaching the lower parts before the break-up has occurred. The maximum spring discharge has reached 42 500 sec-ft., as compared with a minimum flow of only 200 sec-ft. The discharge, during the summer flood of 1917, following a rainfall of 5.42 in. in 15 hours, was estimated at 125 000 sec-ft. and caused a rise of water of 30 ft. with losses estimated at $1 315 000. The Quebec

Streams Commission has considered a number of reservoirs for flood control on this river.

The flood control of the Chateauguay River is also being studied by the Quebec Streams Commission. This stream is another example of a river flowing north in which spring floods are caused mainly by an ice blockade in the lower part of the stream. The remedy proposed consists of creating conditions in the rapid upper part of the stream, whereby a more substantial ice cover will be formed, and to retard and time properly its release in the spring.

General Remarks.—In this discussion, it has been possible to give only a brief summary of the most important flood problems facing the Engineering Profession of the Dominion. It may be of interest to state that, in proportion to population, Canada is spending much more on basic hydrometric work than the United States. Every Province is now covered by the Dominion Hydrometric Survey, the field, office, and publication methods of which are practically the same as those so successfully standardized by the Water Resources Branch of the U. S. Geological Survey.

George M. Lehman,* M. Am. Soc. C. E. (by letter).—More attention should be given throughout the United States to the collection of data relating to the various flood troubles and to physical conditions of affected localities, including both improved and unimproved property, particularly the former.

Municipal and county engineers should be brought to realize the importance of recording at least the probable area and amount of notable local rainfall; heights of water, particularly of crest, at various points; velocities; discharge; duration and extent of land overflow; and the classified character and amount of damage, with its location, and such other happenings as may be of value.

It may not be feasible for all these things to be done by the local authorities, but a great deal could be accomplished without much trouble or expense in connection with the regular duties. After the accumulation of data extending over several years, localities not attempting construction of protective measures on any considerable scale would still receive guidance for general development.

In 1908, prior to the organization of the Pittsburgh Flood Commission and when suggesting that a study be made, the writer, as preliminary to the engineering work with which he later was connected, proposed the following: "To investigate the monetary loss, character of damage and areas affected by the floods in the Pittsburgh District, and secure such other information as may be of value as a basis for further consideration of plans for improvement."

It was further suggested that a part of this study include: (*a*) damage to buildings, equipment, and machinery; (*b*) damage to materials; (*c*) loss to employer by suspension of business; (*d*) loss to employees due to shutdown; (*e*) expense of clearing up; (*f*) charities dispensed and funds for pre-

* Chf., Div. of Waterways, Dept. of Internal Affairs of Pennsylvania, Harrisburg, Pa.

vention of disease; and (*g*) fires controlled through inaccessibility or lack of water pressure. In the classification mentioned, transportation lines, of course, were included.

Because the various interests affected were brought to realize the importance of the matter and generously to respond financially, it was possible to attain the extensive field and office results needed for the local engineering plans. Although the whole work included a drainage area of about 19 000 sq. miles above the city, in that part of the study relating to preventative means, by storage reservoir control, intensive study of special localities above Pittsburgh could not be undertaken as the requisite data were not available. To secure the needed information for all the municipalities would have required much time and a large additional sum of money, only attainable by a widespread and extensive campaign. It was found that reservoir control for Pittsburgh would also render flood reduction benefits to places above, and if each community had had the foresight to accumulate some useful information, the report could have been broadened to include at least many of those communities in a beneficial manner. The findings disclosed that the damage occasioned by the greatest recorded flood amounted to $5 300 000 in direct loss to Pittsburgh.

As illustrative of the importance of knowing conditions, a study was recently made of a part of a river valley in Pennsylvania, having extensive valuable flat land in which is located a city of importance. The writer found that in a 17-mile reach, the area involved by the greatest recorded flood amounted to about 6 980 acres. The width between flood flow lines averaged about 3 400 ft., whereas the width of normal water, between banks, was about 700 ft. Protective measures by dikes was considered as the only means available, as given in a report in 1913 by an Army engineer. These works, the recent study disclosed, would reduce the affected area about 66 per cent.

Many features of vital importance to present and future generations were involved in this particular study. The increase to property values, alone, would be enormous with the flood menace, at least, largely under some form of control.

The State should accumulate the data collected and recorded by the municipal and county authorities. The advantage of such an arrangement is manifest. For a number of cases, special service would be rendered by a State representative.

W. H. Breithaupt,* M. Am. Soc. C. E. (by letter).—The peninsula of Southwestern Ontario is bordered by Lake Huron and its extension, Georgian Bay, on the west and north, Lake Erie on the south, and Lake Ontario on the east. These large bodies of water form, in a manner, a meteorological barrier, mitigating if not arresting undue atmospheric disturbances. The level surface and its cool, equable temperature induce atmospheric conditions nowhere equalled on the land.

In Ontario, precipitation and, to some extent, general weather records extend back about seventy-five years. The Canadian Meteorological Service was

* Cons. Engr., Kitchener, Ont., Canada.

organized in 1870, but observations had been made, to some extent, for a considerable time before. The average annual precipitation, rainfall and snowfall, has varied from about 30 to more than 40 in. In the high altitude region, in the northerly part of the area, the snowfall is heavy, attaining ordinarily 120 to 130 in.; one record (1879) is 153.8 in. The general average of precipitation has varied in cycles, up and down, but there appears to have been no fundamental change since the beginning. Since the first observations, there is no record of such extreme precipitations as have occurred several times during this period, in the region beyond the lakes.

Most of the heavy precipitation in the peninsula has its origin in the Gulf of Mexico. Consider the extreme rain storm that caused the floods of March, 1913, along the Miami River and its tributaries: In the area of maximum precipitation, in Eastern Indiana and Western Ohio, the total rainfall exceeded 6 in. prior to March 24th. The maxima from the beginning of the storm to the dates given were at:

Wooster, Ohio	To March	26th	9 in.
Upper Sandusky, Ohio	" "	26th	8.44 "
Bangerville, Ohio	" "	27th	9.5 "
Bellefontaine, Ohio	" "	27th	11.16 "
Marion, Ohio	" "	27th	10.6 "

In Cleveland, Ohio, on March 25th, 1913, the rainfall was 2.88 in.

At observation stations in Ontario, north of Lake Erie, the records for March 23d to 27th, 1913, are as given in Table 7.

TABLE 7.

Station.	Location.	March 23d, 1913.		March 24th, 1913.		March 25th, 1913.		March 26th, 1913.		March 27th, 1913.		Total water, in inches.
		Rain, in inches.	Snow, in inches.	Rain, in inches.	Snow, in inches	Rain, in inches.	Snow, in inches.	Rain, in inches.	Snow, in inches.	Rain, in inches.	Snow, in inches.	
Port Stanley	Lake shore	0.35	.....	1.64	.....	1.77	.....	0.08	0.2	.....	2.2	4.08
Port Burwell	"	.....	.....	1.55	.....	1.50	.....	0.15	.....	.....	3.0	3.50
Port Dover	"	0.37	.....	1.29	.....	1.19	.....	0.30	.....	0.37	2.6	3.78
London	22 miles inland.	.....	.....	0.92	.....	1.34	.....	0.13	0.5	.....	2.5	2.69
Woodstock	30 "	0.66	.....	0.05	.....	0.64	.....	0.02	.....	.....	3.3	1.70
Guelph	50 "	.....	.....	0.28	.....	0.10	.....	.....	.....	.....	3.0	0.68

It will be noted from Table 7 that the precipitation is higher on the lake shore than inland, that some of it is snow, and that it is much less than the precipitation south of the lake. The totals for 1913, at the stations given, were as follows: Port Stanley, 41.8 in., Port Burwell, 36 in., Port Dover, 41 in., London, 43.6 in., Woodstock, 26.1 in., and Guelph, 28.5 in.

On the general question of the effect of forestation on stream flow, the writer will cite his personal observation of a smaller river for the past fifty years. It is held that for extreme conditions of both flood and low water, the influence of forested surface is detrimental, that forests neither retard

the melting of snow, nor prolong the run-off. The most notable proponent of these views was the late Brig.-Gen. H. M. Chittenden, M. Am. Soc. C. E.* Although for the special conditions in the Rocky Mountain region where, above the forestation line, enormous snowdrifts gather in the great clefts, and where, on the Pacific slope, there are dense forests of lofty evergreens—abnormal as compared to any other part of the Temperate Zone—where 12 in. of an 18-in. snowfall will lodge in the foliage exposed to quick melting by the "chinook" winds peculiar to that region, forestation may increase and bare areas retard the intensity of run-off, this does not hold, in general, elsewhere.

The influence of normal forestation on smaller rivers is so well established as to be beyond question. The Ontario Peninsula was a densely wooded area of general forestation. All observers agree that the floods are greater and the dry-period flow much less than before deforestation, settlement, and the general clearing and farming of the land. Former springs and even brooks have disappeared with deforestation. Rivers which, in the early days, were routes of canoe navigation, are now small streams during the greater part of the year. For instance, the Humber River, with its outlet at Toronto, was on the main route from the western part of Lake Ontario to the Northwest. At present, except a few miles above its outlet, this river is now not navigable for canoes after the spring freshets.

All the rivers of the Peninsula have a much smaller summer flow than formerly, and greater floods in the spring. The Grand River drains essentially the central part of the Peninsula, and its drainage area on the plateau contains more than 400 sq. miles of swamp area. The entire water-shed of the river was originally heavily forested, the swamps also being covered with dense growth, mostly of cedar and tamarack. The high-water channel of 40 or 50 years ago, overflows almost every spring with the snow-melting and the breaking up of the ice. The ice is forced aside over the banks, and cuts at the trees which could not have grown under present conditions. The extreme flood which generally subsides quickly, leaves the heavy ice formation high on the banks for weeks; even with the light snowfalls of the winters of 1921 and 1922, this overflow and ice shove occurred. As to snow-melting the observance in Ontario is that snow disappears in open fields long before it does in the woodland. The head-water swamps were particularly valuable as snow retainers and great, spongy, natural reservoirs. Coincident with their drainage during the past 30 years, the summer flow of the river was reduced to its present low level. This swamp drainage was, mistakenly, assisted by the Provincial Government, and thus a valuable asset was destroyed. Above Bridgeport, the summer low water is now generally about 35 sec-ft., and in the flood of 1912—the greatest on record—the flow was from 19 000 to 20 000 sec-ft.

The great river of Canada, the St. Lawrence, with its unique and superb natural flow regulators, the Great Lakes, has little variation of flow throughout the year. Local floods are caused by ice damming, and local lower water, as in the Niagara River, to long continued, strong east winds piling up the water

* *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 245.

in Lake Erie. An extreme flood on any component pouring into the Lake will cause only a minute difference in lake level, in head of outflow, and, therefore, in rate of outflow.

Relative to brooks and springs: In the County of Waterloo, with an area of about 600 sq. miles, there have been in operation during the course of its history, 112 water powers,* which supplied power mostly to saw-mills and grist mills. Only about 30% of these water powers continue to-day. Few saw-mills are left in the county, as most of the saw logs have been used long ago. The brooks which supplied some of the smaller water powers, are now dry for a great part of the year. Innumerable springs throughout the county have gone dry, many of them within the writer's own recollection.

On the Grand River, exceptional natural facilities exist for water conservation. This river is comparable to the Genesee River in New York State. It has a drainage area of 2 600 sq. miles, and is wide at the upper part, narrowing farther down stream. It rises within 25 miles of Georgian Bay and has a total fall of more than 1 100 ft. Its length, along its windings, is more than 160 miles and it empties into Lake Erie. The tributaries are the Conestoga from the west, rising on the plateau, the Speed from the east, and, lowest, the Nith from the west. Along the main river are three fast-growing cities and a number of towns, and the entire water-shed is one of the most prosperous sections of Canada. Extreme floods occur only in the spring and always come with snow melting on the head-water area. Heavy rainfalls may cause a rise of 6 ft. or more in as many hours, owing to the quick run-off, but these floods do not overflow the main channel. The small low-water flow, which has greatly diminished existing water-power developments on the river, is a sanitary menace and is generally detrimental.

At the foot of the upper river, at Elevation 1 150, there is a basin which, closed by a comparatively low dam, will contain from 2 750 000 000 to 3 000 000 000 cu. ft. of water. Above the site is the heavy snowfall area, and a drainage of more than 450 sq. miles. The basin would be filled every spring and replenished during the summer by the rainfalls. Further storage is practicable on the Conestoga where, with a drainage area of about 160 sq. miles above it, one dam will store more than 1 000 000 000 cu. ft. This will give more than 4 000 000 000 cu. ft. of storage capacity, sufficient to eliminate flood damage on the entire river below the dams and to give a minimum flow of 350 sec-ft., or more—eight to ten times the present low water on a large part of the river. The existing water-power developments would be greatly benefited and additional ones would be economically practicable at a number of places. Further, the conserved water, now destructively wasted, could be diverted and dropped over the Niagara escarpment to the level of Lake Ontario with a development of 15 000 h. p., or more. In a short time, the cities along it will have to depend on the river for their water supplies, if not directly on the contemplated storage basin which would give a much better water, both industrial and potable, than the present low summer flow of concentrated hardness and contamination.

* Investigation by Mr. E. W. B. Snider.

ADOLPH F. MEYER,* M. AM. SOC. C. E. (by letter).—The economic justification for the large expenditures that these great flood-protection projects have entailed has not been thoroughly discussed. Descriptions have been given of towering retaining walls, mountainous fills, miles of bank revetment, thousands of feet of diversion weirs and spacious by-pass channels, and the millions of dollars in assessments paid for the construction of these gigantic works have been cited. To preserve the balance, should not the millions which these projects will return to the investors, have been pictured in equally bold relief?

Each drainage basin subject to floods must be studied as a problem in itself. The gravest danger lies in wrong applications of good solutions. Reference has been made to the general opposition to reservoir construction during the early development of the Miami Project. In other communities, similar opposition is voiced against levee construction, on the Lower Mississippi, for example. Political spellbinders who picture the gathering of flood waters into tremendous reservoirs for later beneficial use in power development and in the increase of navigable depths in streams, are applauded by audiences that give an unsympathetic ear to the engineer who points out that the structures required to utilize flood waters available only once in 5 or 10 years will never return interest on the investment, and that storage for flood prevention and beneficial use are essentially antagonistic.

Engineers are primarily responsible for the false notions which the general public possesses regarding most flood problems, because they too often fail to emphasize why, on one stream, the correct solution of the flood problem is not necessarily applicable to other streams; why automatic retarding basins are the best solution in one case, levee construction in another, cut-offs and by-passes in another, and manually controlled reservoirs in still another. If engineers would mould public opinion on such vital questions as flood protection, the writer believes they should place greater emphasis on these phases of their projects for the benefit of the public.

The cost of every flood-protection project must be justified to those who pay for it. The comparative cost and accomplishment of the projects do not stand out as prominently as they might. The fact that communities in Ohio have spent $35 000 000 for the control of floods on a water-shed of 3 500 sq. miles is not *prima facie* evidence that communities in Minnesota can afford to spend $2 000 000 for the control of floods on a basin of 16 000 sq. miles.

The protection of agricultural lands is one of the flood problems in Minnesota. A typical example is that of about 100 000 acres of rich bottom-lands in the Minnesota River Valley. Some of these lands are flooded, on an average, every 2 or 3 years, others less frequently. Having determined the extent and frequency of flooding from the best available sources, this problem resolved itself into determining how much protection offered the greatest return on the investment and how this protection, if worth while, could be most economically secured. Although these studies have not been fully completed, the answers are:

* Cons. Hydr. Engr., Minneapolis, Minn.

First.—It does not pay to protect agricultural lands from crop losses through rare floods, that is, floods that will not recur, on an average, once in about fifteen years.

Second.—When lives, farm buildings, city property, or other important improvements are not endangered, it does not pay fully to protect the flood-control structures themselves against floods unlikely to recur once in a century or more.

Third.—Protection of these agricultural lands must be secured through the construction of manually controlled storage reservoirs, combined with the provision of reasonable bank-full carrying capacity in the channels below.

A full statement of the reasons for these conclusions cannot be undertaken in a short discussion, but the writer will state that the cost of levee construction and channel improvement cannot be justified by the crop savings affected, even if there were no other objection to these methods of flood protection on this stream.

Every large tributary of the Minnesota River flowing down steep slopes from the uplands into the valley of the main stream, has either succeeded in blocking the stream sufficiently to form a lake or to produce a long flat reach above the point of debouch. These lakes and the longest reach of the stream where the slope is flat, have been selected as sites for three reservoirs.

Below the proposed reservoirs, the stream has steeper slopes and relatively larger capacity, that is, from one-sixth to one-fourth of the capacity necessary to carry off, at bank-full stage, a 25-year flood. At the extreme upper end of the valley, the channel will carry only about one-twentieth of such a flood. For this reason, flood water stored in the upper reservoir must be discharged over a period of several months, in wet years. This fact alone shows the impracticability of automatic retarding basins for this stream. The smaller the drainage area the larger the ratio of the flood flow to the bank-full capacity, hence, the greater the required storage reservoir per square mile of tributary drainage area.

The Minnesota River reservoir farthest down stream will be filled most frequently, but may be emptied again in a few weeks. In fact, this reservoir comes nearest to having the characteristics of an automatic retarding basin. Even this reservoir, however, must be closed off entirely at times, for a day or two, in order to give the single large tributary which enters just below, the right of way in the main channel.

The writer believes it may be set down as a fundamentally correct principle, applicable not only to the Minnesota River, but to most other streams, that both the length of time over which flood water should be detained and the quantity of water which should be detained per square mile of tributary drainage basin for flood protection purposes, decrease down stream. The flood-plain of most large streams, particularly in the lower reaches, affords a detention basin of no mean proportions. Levee construction preserves and enlarges these natural basins.

In conclusion, the writer would reiterate that he believes an endeavor should be made to emphasize more strongly the basis on which each flood-

protection project is economically justified, and the factors that in each case made the solution adopted the best one, if engineers would prevent the formation of prejudices, both within and without the Profession, for and against all methods of coping with this most vital problem.

JOHN MILLIS,* M. AM. SOC. C. E. (by letter).—Under date of June 20th, 1893, the writer, then stationed at New Orleans, La., in charge of the Fourth District, Mississippi River, extending from near Vicksburg, Miss., to the Head of the Passes, proposed to the Mississippi River Commission that certain examinations and surveys be made with the object stated as follows:

"In general terms, the object of the proposed surveys and examinations is to determine if possible whether there is any practicable method or device, or any modification of existing methods or devices, which would be likely to lessen the danger and losses resulting from the excessive and prolonged flood heights to which the lower river is now subject, and if so what would be the probable cost of carrying out such plans or methods as may give sufficient promise of practicability to warrant their serious consideration."

In response to a request from the Commission for a more definite statement as to the reasons for the studies proposed, the following was submitted in a letter of June 27th, 1893:

"From general observations extending over the period of my service in charge of this office (from fall of 1890), from conversations with citizens, and from the opinions expressed and discussions in the public press, I have become satisfied that there is a growing apprehension among the residents of the lower valley as to the ultimate results of the completion of the levee system, and an uncertainty as to the outlay which will be necessary to make the levees of the lower river safe when those above are completed, and as to whether this outlay will be within the limits of justifiable expenditure in protecting the interests involved.

"Whether warranted or not it is undoubtedly true that these doubts and apprehensions have been greatly increased by the experience of the last few weeks and that they are no longer confined to fanatics and alarmists."

This recommendation was duly approved and an expenditure necessary to make the proposed surveys and studies was authorized. A detailed report was submitted under date of June 24th, 1894. The following are extracts from that report:

"With the information and experience available up to date I think the following can now be accepted as established either beyond question or at least beyond reasonable doubt:

"1. There has never yet been a considerable flood which has not resulted in one or more disastrous crevasses below Red River, and in every case where a crevasse was of considerable size and remained open it has had an immediate effect in lowering the surface of the river and an effect in restricting the subsequent height of the water until the flood subsided. The extent and duration of these effects have depended not only on the location and size of the break, but also on the period of the flood at which it occurred.

* * * * * * *

"The grades adopted for the new work built under the recent revival of levee renewal and restoration have been regarded as provisional * * *. While it may be possible to compute with reasonable accuracy the heights

* Col., Corps of Engrs., U. S. A., Chicago, Ill.

which any past flood would have produced at points below Red River had the levees held, there are many uncertainties connected even with this problem, and as to the determination of the limits of the flood heights for the future, that is believed to be absolutely impossible.

"Many of the elements involved are entirely indeterminate and most of them extremely uncertain. Among these elements are the climatic conditions which determine the total amount of water to pass out of the valley, and its distribution both geographically and in respect to time. These there can never be any means of calculating beforehand. The nature and extent of the effects of breaks that may occur above and of violent storms like that of October, 1893, are also beyond prediction. In addition to these influences there are certain others at work which are certain to have an important effect on the lower river. While these effects cannot be pre-determined quantitatively there is little room to doubt their tendency to increase flood heights.

"The most important of these is the restoration of the levee system; others are the constant increase of the cleared and cultivated areas within the limits of the valley and the extension of artificial drainage. It is true that these influences which tend to increase flood heights seem to be in a measure compensated by an increase in river section, but this is only one effect of the cause which it is desired to remedy, and the effect cannot be looked to as a remedy for the cause which produces it, since increased section is only produced and maintained by increased height of the water during flood seasons.

"3. The present tendency, therefore, is gradually towards increased height and section of the levees and the final limit of this height and section cannot now be determined. With the increased height of the levees and floods comes also the increased risk, since the immediate damage resulting from a crevasse increases very rapidly with the height of water above the ground surface at the locality of the break. The chance of success in closing a break during the flood becomes correspondingly less, while the cost of closing, if closure is at all possible at that time, and the expense of permanently restoring the broken levee after the water has subsided, both increase in a similar accelerated ratio.

"The effect on the annual expense of maintaining the levees must also be considered. The work of maintenance consists in the repair of damages caused by rain wash, wave action, and leaks, in keeping the embankment cleared of injurious vegetation, and in renewals where rendered necessary by caving banks. All of these are on the increase with increasing heights.

"The work of raising the levees is necessarily progressive. The necessity of building to a higher grade has been several times demonstrated and different 'provisional grades', each higher than the preceding, have already been adopted. The raising of the provisional grade has each time been done before more than a comparatively small portion of the system was brought to the preceding grade. Assuming that the levees will be completed to the present provisional grades before another flood shall require a still further increase in height, it is quite certain that another increase will eventually be necessary, and it is not at all probable that such increase could be applied to the entire length of some 400 miles of levee in a single season. The work would have to be done as heretofore by successive stages and as the resources available might permit.

"It therefore results that the levee system has never yet been and is not likely to be for a considerable time to come in what may be termed a consistent condition. It has always comprised portions of various standards of height and strength of section, and while incomplete and in the process of construction it has been exposed each year to the chances of a strain which it was known was far greater than the weaker portions could stand, with no provisions to relieve or protect those weaker portions until they could be brought to at least an approximation to a uniform standard.

"As a consequence the structure has always yielded at some one or more of the many weak places, at points which could not be predetermined and where preparations for such an event were therefore not made. The results were necessarily disastrous, but such breaks have invariably relieved the strain at other points and limited the damage to a greater or less area in the vicinity of the break.

"The present indications are that this is likely to be the condition of affairs at least until all the levees are brought to the last adopted provisional grade, even if a higher flood should not require another elevation of grade before that time, and there is very good reason to anticipate, as heretofore explained, that the process may continue indefinitely.

* * * * * * *

"These premises lead to the conclusion that the present and immediate requirement is some means to protect the structure against exposure to strains in excess of its resisting strength, while utilizing it to the extent of its resisting capacity, during the process of completion; and this need exists irrespective of any plans for establishing a definite limit to flood heights as a permanent effect for the future.

"That such relief is afforded immediately and effectively by breaks in the levees at high stages of the river is well known to everybody who has observed a flood below Red River, and that no appreciable relief is produced, so far as past experience and observation go, by an opening of considerable depth and limited width which remains constantly open is equally well established.

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"In respect to a sudden break or cut in the levee made at a very high stage of the river and left open till the flood subsides or the period of danger is over, there is scarcely a single effect which can not now be determined with at least reasonable certainty beforehand from actual experiences with crevasses. It only remains to determine whether the amount of the risk now uninsured and the degree of relief and protection which cutting the levee at high water affords will justify the outlay necessary to prepare beforehand one or more places where cuts may be made without such injury to other interests as to condemn the plan as a measure of security.

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"Before proceeding to the description and estimates of the relief openings to be proposed, brief reference should be made to the discussion of the subject of the relations between gauge heights and rates of discharge, and the deductions which have been made as to the comparatively slight effects on the height of the river surface to be expected even from openings of very large section and rates of discharge. Calculations have also been made to show that the entire discharge through a large crevasse would elevate the surface of the river but little if the discharge were poured into the river instead of running out of it.

"While not prepared to attempt a complete analysis of this question I believe these deductions are considerably in error so far as the lower river is concerned. The rapidity with which the effect of a break was transmitted to distant gauges and the promptness and degree of its influence in lowering the river surface, even before the break was more than a few feet wide, was quite remarkable in the case of all the important crevasses during the last few years.

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"Below the mouth of the Red the depths are great, the slope gentle, and the area exposed to overflow beyond the river banks is small, being confined to the bottom lands on the east side above Baton Rouge. It seems more nearly correct to regard the effect of a break as resembling that of tapping a large reservoir, rather than the taking off of the top of the flood in the sense of diverting a part of the flowing stream.

"In general the form of opening proposed is to resemble a crevasse in many respects, but it is to have the important differences that its location is predetermined, its time of occurrence is to be under control, and limits are to be set to its width, depth, and the area overflowed. It is to be opened only in case of necessity and is to have an overflow area or basin as large as possible which is to be brought as near as possible to the opening itself in order to afford the freest possible discharge. After being opened during flood it is to be closed the succeeding low water and not opened again until it may become necessary. The area overflowed, or a part of it, is to be cleared of all vegetation as far as possible and kept cleared, and provision is to be made to prevent the interruption of roadways while the levee may be open.

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"Some form of gate could no doubt be devised which would permit of opening or closing the crevasse at will, but the difficulties are many and the expense would be large, so that a definite plan of this kind is not now presented."

It was proposed to prepare two places, both on the left or north bank of the river, where, in case of necessity, the levee could be opened to relieve an extreme high stage of the water. One of these was to be near the Lake Borgne Canal, 10 miles below the lower limits of New Orleans, to discharge into Lake Borgne. The other was planned at Kennerville, La., about 10 miles above the city, to discharge into Lake Pontchartrain. The site of the old Bonnet Carré Crevasse, 30 miles above New Orleans, was considered, but was rejected for reasons already explained. Under present conditions, studies would be made of permanent masonry relief openings with gates under complete control, which would admit of closing as soon as the stage of the river permitted, without danger to the levees, and the advantages of the Bonnet Carré site would be thoroughly worked out. Estimates of cost, of course, would be radically revised.

In the paper by J. A. Ockerson, Past-President, Am. Soc. C. E., reference* is made to the oft quoted Nita Crevasse of 1890 as a demonstration of the futility and detrimental effects of "outlets." The views expressed on this question and the consistently adhered to policy of the Mississippi River Commission are unquestionably sound, but it should be clearly understood that Col. Ockerson is referring to uncontrolled outlets. He does not discuss safety valves, relief openings that are to be brought into operation only when the pressure reaches or approaches the danger point, and that can be closed as soon as that point has been passed for the time being.

There is no intention on the part of the writer to pose merely in the attitude of "I told you so"; this rarely proves of constructive assistance in solving any great problem or carrying out any great work, nor should there be attributed to this discussion a motive of disparagement of the formidable and commendable work which the Mississippi River Commission has so ably and faithfully carried forward, as outlined in Col. Ockerson's paper. What study was given by the Commission to the previously mentioned suggestions at the time they were submitted and why they were not regarded as worthy of favorable consideration is not known by the writer, but it must not be overlooked that those in authority also have the responsibility, and, presumably, they have a wider vision than subordinates, especially respecting questions of policy

* See p. 1464.

and expediency. In the event of a failure, the Commission would have become responsible, not the proposer of the new plan.

It is true, however, that since Col. Ockerson's paper was published, the Lower Mississippi Valley has again suffered greatly from a higher stage of the lower river than was ever known before, with the oft-repeated story of breaks in the levees, large areas overflowed, menace to human life, and great losses and damage. The same experiences have been repeated during the 28 years that have elapsed since the report referred to was written, and more than once.

A large investment has been made in works to control the Lower Mississippi River. The human interests and the property values involved are vast, and the efforts to effect complete control and security are still much short of satisfactory consummation. Has not the time arrived to consider seriously, first, whether the greatest possible return in confidence, insured safety, and removal of apprehension is being derived from this investment; and, second, whether it would not be practicable to provide a greater measure of security and to insure more effectually against such widespread loss and disaster as the lower river region has recently suffered.

C. McD. Townsend,* M. Am. Soc. C. E. (by letter).—The Yellow River has been used for many years as an argument to prove that levee construction and deforestation raise the bed of a river. Heretofore, the descriptons of its flow have been chiefly derived from the reports of missionaries and soldiers who could have only a cursory knowledge of river hydraulics. The paper by Mr. Freeman on "Flood Problems in China", which is based on instrumental surveys, is, therefore, a valuable addition to hydraulic literature. Its publication at the present time is also most opportune, by reason of the recent flood in the Mississippi River, and the discussion throughout the United States that such an event invariably invites.

The Yellow River as described by Mr. Freeman has a flood discharge about equal to that of the Miami River,† and less than one-half of the estimated maximum discharge of the Missouri River, which it resembles in many features.

The Missouri River has an average slope of 8 in. per mile near its mouth, a length of about 2 500 miles, and flows through the bad lands of Montana, where its waters become saturated with sediment, just as the Yellow River carries down the loess of Shansi. The quantity of sediment carried by the Missouri during flood stages has never been determined, but observations made in 1879 at St. Charles, the gauge reading 23 ft., and the extreme flood stage being 39.5 ft., give a silt burden of 2.7% by weight‡ as compared with 0.4% at low water and 6.5% at flood stage in the Yellow River.

The discharge observations submitted in Fig. 17, translated into English units, give for the Yellow River when rising, a cross-sectional area of 4 252 sq. ft. at an elevation of the water surface of 124.6 ft., a mean velocity of 3.14 ft., and a discharge of 13 383 sec-ft.; at an elevation of 127.0 ft., a cross-sectional area of 6 781 sq. ft., a mean velocity of 4.46 ft., and a discharge of

* Col., U. S. A. (*Retired*), Brooklyn, N. Y.

† Report of the Chief Engineer, Miami Conservation District, Vol. 1, page 26.

‡ Report of Chief of Engineers, U. S. A., 1881, p. 1649.

30 254 sec-ft.; and, at an elevation of 140.1 ft., a cross-sectional area of 32 708 sq. ft., a mean velocity of 8.2 ft., and a discharge of 270 650 sec-ft.

The observations on the Missouri River at St. Charles in 1879 give at 6.3 ft. on the gauge, the lowest stage observed, a cross-sectional area of 8 610 sq. ft., a mean velocity of 2.79 ft., and a discharge of 24 060 sec-ft.; at a 9.3-ft. stage, a cross-sectional area of 12 540 sq. ft., a mean velocity of 3.77 ft. and a discharge of 47 230 sec-ft.; at 20.6 ft., a cross-sectional area of 31 714 sq. ft., a mean velocity of 6.67 ft., and a discharge of 211 450 sec-ft.; whereas, at the highest stage observed (23.6 ft.), the figures are 41 060 708 sq. ft. and 290 800 sec-ft. The discharge section on the Missouri River appears to have been in a straighter portion of the river than those selected on the Yellow River. This would have an effect both on the cross-sectional area and the mean velocity, as will be explained later.

The ratio of high water to low water is somewhat smaller on the Yellow River than on the Missouri, and very markedly smaller than that on most of the eastern tributaries of the Mississippi. The valley of the Missouri River is contracted by hills, so that its slope resembles that of the Yellow River between levees, but when the waters of the Missouri debouch into the Mississippi Delta below Commerce, Mo., the alluvial valley assumes slopes of about 1 ft. per mile in sections at right angles to the river bed. Farther down stream, steeper slopes are frequently found. Most of the river bars above the mouth of Red River have also a higher elevation than the land surface at the foot of the hills which limit the delta.

The writer is one of those who has maintained that the rise of a river bed below the area of deposition, caused by its confinement between levees, was so small that it could be ignored in levee construction, and he has also questioned the statement, so often reiterated, that deforestation has an injurious effect on the discharge of a river. He has based his opinion on the observations made on the Po and Rhine Rivers during a period of 100 years. He has examined, therefore, with considerable interest the cross-sections submitted by Mr. Freeman.

In Cross-Section No. 2, Plate XIV, the low-water surface of the river is about 5 m. above the surface of the ground outside the North Inner Dike and 1 m. above that on the outside of the South Inner Dike; in Section No. 4, the corresponding depressions are 3½ m. and 3 m.; in Section No. 5, the ground outside the North Dike is 2 m. below the low-water surface, whereas the ground beyond the South Dike is above it for a distance of 5 km. For a distance of 120 km. below the point where the Yellow River debouches from the mountains into its alluvial plain, the average rise of the low-water plane, therefore, has been less than 3 m. since levee construction was begun. Whether this rise has been due to the confinement of the river between levees, to deforestation, or to a change in the river slopes, has not been determined.

In his description of ancient Chinese flood protection, Mr. Freeman states that the Chinese have had to contend with the problem of flood control for a period of 4 000 years, and that the great hydraulic engineer, Yu, lived about 3 000 years ago. If the Yellow River has been leveed for 4 000 years, or even if the levees owe their existence to Yu, the rate of rise of the river bed in the upper

parts of the Yellow River Delta is less than 1 m. per 1 000 years, an amount considerably less than the rise of the bed of the Po, as estimated by Lombardini, or of the Rhine, as computed by certain German engineers.

To provide for errors of computation, the Mississippi River Commission constructs levees, the crowns of which have elevations 3 ft. above the highest flood, and if the waters of the Missouri which is the principal source of sediment in the Mississippi, have no greater power to raise the river bed than those of the Yellow River, the factor of safety thus introduced will render the levees of the Mississippi River safe from the effects of deforestation and levee confinement until 2922 A. D., instead of 2022 A. D., as the writer has contended.

In a book on the "Hydraulic Principles Governing River and Harbor Construction", now in press, he has also suggested that a rise of the bed of the Yellow River between the diversion of 1852, and Honan, can be accounted for by a change in slope as the river debouches from the mountains into its delta, and to an effort on the part of the river to equalize its slopes in the angle thus formed. The irregularities in the surface of the ground, outside the levee lines, would indicate that a force was operating before levees were constructed, similar to that which has caused a fill between them in more recent years. He has also suggested that below the diversion, the bed of the old river could have filled during floods occurring between the date of the cut-off and the construction of a new levee line, just as the Missouri River fills its old bed when it creates a cut-off. The permanency of the steep slope in the vicinity of the diversion does not conform, however, to the usual effect of cut-offs in alluvial streams. There is usually a rapid recession of the section affected by the steep slopes. The Napoleon Cut-Off in the Missouri River affected the regimen of the river for 20 miles above it, and even greater changes were noted after the water-proof cut-off in the Mississippi River above Natchez, Miss., occurred.

Mr. Freeman, moreover, does not appear to place much faith in the theory that levee construction raises the river bed dangerously as he proposes to substitute for the existing levees interior lines which will confine the flood discharge to a width of 1 640 ft. So radical a contraction will unquestionably produce scour during flood stages, but it will also produce a marked increase in flood heights.

In their passage through the loess of Shansi, the waters of the Yellow River have become saturated with sediment, carrying from 6.5 to 10% during floods. Water with such a burden will move the material it scours from the river bed as sand waves, or in intermittent suspension, and, in either case, its rate of transmission will be slower than that of the water. In the Mississippi River, the motion of a sand wave is about 40 ft. per day when the river currents are 6 ft. per sec. In the Sacramento River, it is about 4 miles per year. Whenever the river currents are retarded, the sand waves move more slowly, and material carried intermittently in suspension is deposited. The result is that in a river carrying a large quantity of sediment, a flood discharge scours its pools and deposits the material thus moved on the bars between them. In the Mississippi River, it has been found that the rate of rise of some of the bars equals one-

half that of the water surface. M. Girardin reports that the rise of bars on the Rhine is at the rate of 1:5, and the writer has found a similar rise of bars in the comparatively clear waters of the Upper Mississippi. The ratio of the rise of the bars on the Missouri River has never been accurately determined, but it has been stated that during the great flood of 1844, river pilots reported channel depths of only 9 ft. During a falling river, the material deposited on a bar is scoured into the pool below it. The movement of the scoured material, therefore, is intermittent.

It is noted that Mr. Freeman has submitted cross-sections of pools. It is the elevation of the bars, not the depth in pools, which determines the height of a flood. Flood heights on the Mississippi River have increased more than 10 ft., both at Memphis, Tenn., and Lake Providence, La., and more than 6 ft. at New Orleans, since levees have been constructed along the river, and the increase in flood heights below Arkansas City during the recent flood can be accounted for by a further confinement, resulting from the closure of a waste weir in the levee line at the upper end of the Tensas Basin. During the floods of 1912, 1913 and 1916, the flow through this opening was about equal to the flood discharge of the Yellow River.

The straightening of a non-tidal river, as proposed by Mr. Freeman, will not prevent the formation of bars. It merely introduces an element of uncertainty as to the locality at which they will form. It lowers flood heights at one locality only by raising them at another. In a river with a proper curvature, the bars form in the crossings, rising and scouring out with variations in the discharge; but, in a straight river, they change their location with every flood.

Mr. Freeman suggests laboratory experiments as a means of increasing knowledge of river hydraulics. Experiments on the rivers themselves afford a much better means. Engineers of recognized ability have been building levees and improving rivers in Europe for hundreds of years, and have given exhaustive descriptions of their successes and failures. The knowledge thus acquired should be utilized. Girardin describes the rise of the river bed and the increase in flood heights that accompanied the straightening of the Rhine River through the Canal de Mirabel. De Mas gives, as an example, the straightening of the River Theiss, and the resulting disastrous flood at Zegedin in 1879. The Austrians who relied on the flood discharge to deepen the Danube River in the project of 1882, adopted the curved trace in the project of 1899. Jasmund's project for a further improvement of the Rhine River, also provides for a curved trace similar to that adopted by the French on the Rhone and the Italians on the Po.

It is evident from reading the paper that the cause of "China's sorrow" is not a raising of the river bed, but a caving of the river banks, which destroys the levees located near them, and it is because of this caving that the Chinese have left such a space between the levee lines on opposite sides of the river. It was also the determining factor in the original location of levees along the Po, the Rhine, and the Mississippi Rivers. The contraction of river channels by levees was only undertaken by the German and Austrian

authorities after a satisfactory method of bank protection had been invented. Jasmund describes conditions along the Rhine in former days, which read like extracts from this paper, and there are similar descriptions of the Po River. On both rivers, dikes pointing down stream were first used to prevent bank caving, similar to the Chinese groynes. Jasmund states that such dikes failed on the Rhine and were replaced by triangle "werke" which were created by connecting the outer end of one dike with the inner end of the next one below it. When this method of protection also failed, river straightening was resorted to, and bank revetment was abandoned for many years. When levees then caved into the river, villages were moved away from the river bank, and levee loops constructed. In other words, the Chinese of to-day have about the same knowledge of river hydraulics as was possessed by the nations of Europe prior to the Napoleonic Wars.

However, during the past century, there has been a marvelous development in all branches of Engineering Science, both in Europe and the United States. The engineer who would solve the problems arising to-day must be familiar with the history of our own times. This is also true of river hydraulics. The proper principles governing the improvement of non-tidal rivers have been evolved during the past 25 years.

The subject of bank protection has been investigated by French and German engineers for many years. Schlechting sums up the results of the experiments to his time on German rivers, and his recommendations have been adopted with success, not only by Germany, but by France, Italy, and Austria. There is also an American adaptation of an old Italian invention, that merits attention in rivers carrying a large quantity of material in suspension. Various permeable dikes, of which the retard is an example, have been successfully used on Western rivers. Their object is not to divert the current from a bank, but to check its velocity and induce a deposit of sediment. As Mr. Freeman vividly describes in his paper the disasters which have accompanied the Chinese method of bank protection, its substitution for the successful methods used in Europe and the United States, is not considered advisable.

When an engineer constructs an irrigation ditch to carry water flowing with a velocity of 8 ft. per sec., he places a covering of concrete on its bottom and sides to prevent carving, even in glacial drift. The banks of a river flowing in material which it has deposited, are less stable, yet Mr. Freeman would force such a river from the course it is striving to follow into a straight channel, and rely on loose stone piles 300 to 600 ft. apart to maintain it in this constrained position. When flowing water is forced to change its direction, it exerts a pressure against the restraining medium, and if flowing in an open channel, its surface is raised along the restraining bank, and a difference of elevation exists on opposite sides of the channel. The filaments of water then cease to flow in straight lines and have a helicoidal motion which is very destructive to banks. The direction given the Chinese groynes, not only is inducive of the helicoidal flow, but also directs it into the unprotected space between them. The Missouri River Commission made a series of experiments with similar structures which were termed "bank-heads". One of them is now on the opposite bank of the river, several miles from the river channel.

In its efforts to straighten the Missouri River below Jefferson City, Mo., the Missouri River Commission lost 50% of the works of contraction in 10 years. On the Mississippi River, the length lost by a cut-off is regained by caving in about 20 years.

The writer has been exposed to typhoons both in the Philippines and on the coast of China. As a result of his experience, he questions the effectiveness of forestation as a means of reducing the floods of Chinese rivers. There exists in the Philippine Islands the most efficient forest service that can be found in the world, and it is protecting a vast area from deforestation according to the latest and most scientific principles, yet he observed a rainfall of 18 in. in the forests of Benguet, most of which fell in 24 hours, and one exceeding 12 in. per day in the City of Manila. Neither the Bug River nor the Pasig River, appeared to be deterred by forest regulations from overflowing their banks, nor has it been proved that a Chinese typhoon is more amenable to regulations in Chinese hieroglyphics.

Before reforestation can be applied practically to restraining typhoons in such a country as China, it is necessary to know how large a forest will be required to reduce an 18-in. rainfall to such proportions that the streams will carry it without overflowing their banks, and where such forests can be located without interfering with the agricultural pursuits of a delta having a population of more than 1 000 per sq. mile. It is self-evident that the waste land of the country will not be adequate for such a purpose. The transpiration of moisture by a forest or a field of wheat may affect a local summer shower, but a flood results from storms caused by an atmospheric saturation of moisture derived from large bodies of water and carried over the land by winds. A forest covering prevents the excessive erosion of steep hillsides and diminishes the amount of material which is transported into the area of deposition at the foot of the hill. It fails, however, as a means of changing the direction or reducing the intensity of atmospheric currents, but a range of mountains, even though comprised of rock on which no vegetation can grow, will cause a heavy precipitation on the slopes toward which the wind is blowing.*

Thomas H. Wiggin,† M. Am. Soc. C. E. (by letter).—Mr. Freeman has presented a most interesting description of the more important flood problems of the huge delta plain of Eastern China. The information about the Yellow River in particular should be of great interest as it contains the first accurate survey data obtained on that stream.

The writer spent nearly two years in China, studying the canal, flood, and irrigation problems of the regions described by Mr. Freeman, as well as certain other regions farther inland and more southerly.

The Red Cross surveys were undertaken as a result of a disastrous flood in the Huai River District in 1911, in which many lives were lost and crops destroyed over an area of several thousand square miles. Mr. Freeman also mentions the floods of 1921 in the Huai River District and others of which no account has yet reached the United States, except a brief one in the Review

* "The Cyclones of the Far East," by the Rev. José Algué, p. 55.

† Cons. Engr., New York City.

of the Far East. The writer came through part of the flooded district by train in July, 1921, when the water was thought to be nearly at its maximum height and within about 2 ft. of the record elevation. For perhaps 100 miles along the railroad a vast expanse of water, from a few feet to 8 or 10 ft. in depth, spread over the country. The reason more was not heard of this flood was that, unlike the disastrous flood of 1911, it rose gradually, after the harvest. Most of the crops were saved, and few lives were lost. Along the Grand Canal in Shantung, the flood, in the same period, reached exactly the same elevation as the maximum high-water marks of previous floods, which had been pointed out to the writer by the natives a year before.

The writer had a rough estimate made, from the inadequate maps available, of the cost of completing the double (that is, the outer dikes) of the Yellow River from the Peking-Hankow Railway to the sea. The sum required was about $4 000 000 Chinese currency, generally equal to about $2 000 000 U. S. currency. As suggested by Mr. Freeman, it appears to be possible to narrow and straighten the Yellow River between inner dikes, but the writer has proposed as an immediate safety measure the completion of the outer dikes, which will prevent a change of course like that which occurred in 1851, and will give time for work of further improvement as may be indicated by careful study and experiment. One field engineer of the Grand Canal Board, who visited the break of 1919 soon after it occurred, suggested that when the river rises rapidly, it cannot always deepen its channel fast enough to provide the customary channel within and below the tops of the inner dikes and, therefore, overtops these dikes where they are lowest, as at roadways across them. Another engineer believes that the inner dikes sometimes slough on the land side, owing to the deep slopes. These dikes are frequently about 20 ft. wide on top, with slopes of about 1 on 1½. Outer dikes and cross-dikes give a reserve line of defense so far from the rapid current that only a quiet lake rises against them.

The writer learned recently of the heavy crop of spring wheat harvested early in June, 1922, from the border land of the Yellow River, which was flooded by breaks in July, 1921. Missionaries and other foreigners have observed that the flooded land yields abundantly the following season and even two seasons after the flood. The capillary power of the loess seems to be very great. Inundation irrigation, if it could be controlled, would be very beneficial. The writer found many examples of this on the Fen River in Shansi. When the inundation is not controlled, it may be disastrous one year and cause fine crops the next. In any scheme of flood control, irrigation should be considered. Mr. Freeman notes that the Yellow River is only above the general level of the land in the upper parts of the alluvial plain, that is, west of the 1851 break. Even there, it would require digging through 1 km. or more of high ground within the inner dike to get water for irrigation on the land, except in times of very high floods. Along the new course east of, that is, down stream from, the break of 1851, it would be much more difficult, as the river seldom rises above the land. Irrigation from the Yellow River is, therefore, difficult, both on account of the relatively high banks and the tremendous silt content of the flood waters. It is believed, however, as Mr. Freeman

suggested in 1920, that it would be well worth trying above the break of 1851, and experience there might lead to more general use of the Yellow River. The Chinese have made surveys for using it in the upper stretches near the Gobi Desert.

The writer has tried to visualize the phenomena which occur when the Yellow River leaves an old bed and finds another course, lower down, to the sea. Doubtless, at first, a broad lake spreads with rapid deposition of silt away from the main stream. Along the lowest ground, the main body of water pushes ahead toward the sea, and water moving laterally from the main stream deposits a broad ridge of loess sloping away from the main stream on both sides and gradually building up the banks farther and farther down stream. The ordinary bottom of the new stream is little, if any, below the surface of the ground as it existed when the change of course occurred. During floods which come after the new banks are established, the bottom is temporarily lowered, as has been described by Mr. Freeman. The gentle slopes of the broad ridge, the top of which is occupied by the groove in which the river flows, provide the thick banks which afterward confine the stream, except in times of flood. Thereafter, the stream stays in the course, maintaining a slope of about 1 : 10 000, until the mouth is built out into the sea and the stream is correspondingly raised parallel to itself, on the slope mentioned, sufficiently to cause it to drop again out of its bed on land which has become relatively lower. In recent ages, these changes, of course, have been made less frequent by dikes constructed by the people.

The raising process, judging by difference in level of the old and the new dikes in the vicinity of the break of 1851, proceeds at the rate of something like 3 ft. per 100 years. This rate is doubtless controlled by the depth and consequent rate of filling up the sea at the mouth of the river. If a deep sea existed, like the Gulf of Mexico, then jetties like those constructed by Eads at the mouth of the Mississippi, would prevent the rising of the river bed. The China shores, however, are shallow, and advance with comparative rapidity, as can be traced by finding the names of new shore towns in the history of succeeding centuries. A rate of 3 ft. per 100 years, considering the river slope of 1 : 10 000, would correspond to 30 000 ft., or 6 miles per 100 years at the mouth of the river. In the meantime, several other smaller rivers and the great Yangtze which, however, does not carry so much silt, are also building out the shore in other places.

The manner in which the main current finds its way through a lake of thick mud can be observed in the vicinity of Tientsin where hydraulic filling is done by pumping through pipes, to diked areas, the silty water dredged from the river.

The writer traveled for three weeks in the Central Grand Canal, Huai River, and Hungtze Lake region, at the request of His Excellency, Chang Chien. The principal problems there were:

(*a*) Sufficient irrigation water for the great staple, rice.
(*b*) Navigation in the Grand Canal, Salt Canal, Hungtze Lake, and Hsien Nü Miao Canal.
(*c*) Flood channels and reclamation.

The order of stating these objectives is that which seemed to be the order of importance in the minds of the Chinese officials. The pre-eminence of irrigation for the rice is easily understood. The Red Cross plan for the permanent drainage of Hungtze Lake was not regarded with favor for that reason and also because the lake provides storage for navigation. The dry-weather flow of the Huai River is not always sufficient without the aid of storage. At present, Hungtze Lake has huge areas of shallow flowage and a high evaporation loss, so that its net storage effect may not be as large as is thought by the Chinese. Reducing its area and raising its banks appeared to the writer to be the logical solution. E. W. Lane, Assoc. M. Am. Soc. C. E., who has been in this region for several years studying irrigation and canal problems for His Excellency Chang Chien, will have, if the funds are sufficient, the data for a real engineering solution.

To provide a flood channel for the Huai River and prevent the flooding of enormous areas previously mentioned is one of the most important flood-control problems in China. The problem has been made more difficult in and below the Hungtze Lake region by the recent excavation of a flood channel in the Province of Anhwei, causing more prompt drainage of the flooded tributary areas. In China, as elsewhere, every main problem has many lesser ones which complicate the solutions. The flood flow of the Huai River in 1916, as measured by the Chinese Conservancy engineers with current meters and floats, appears to be nearly twice as great as the maximum flow previously estimated by the Red Cross engineers using high-water marks and slope and area computations.

Mr. Freeman advocates a flood channel to the northeast into the ocean. His Excellency Chang Chien recommended a similar route, as stated by Mr. Freeman, the cost being estimated at $45 000 000 (Chinese money), using soldiers at a low wage. The Red Cross engineers proposed to send the Huai flood waters south into the Yangtze where they now go for the most part, but with difficulty on account of deficient capacity of the outlet at Hungtze Lake. Their plan was to cost about $35 000 000, including the drainage and irrigation of Hungtze Lake. The superiority of a separate channel to the sea is obvious as the Yangtze River has floods of its own, and the floods of the Yi and Shu Rivers also (Fig. 22) would be more cheaply cared for if the Huai River flood passed nearer them. However, the writer found that less than $10 000 000 (Chinese money), would apparently provide a diked (not dredged) channel from Hungtze Lake to the Yangtze River through the continuous chain of lakes which lie in that direction. The existence of these lakes, with their large areas of reclaimable land, render the southerly route worthy of careful study.

Mr. Freeman proposes for the Huai River channel to the ocean a novel form of construction (Fig. 23), wherein narrow initial channels are dredged along the sides of the final broad channel and the dredged material is used for channel dikes. Floods are counted on largely to finish the excavation of the central core between the initial channels. The writer knows of one case in North China where the floods excavated a new channel from a narrow cut. The method is often used in making cut-offs. Mr. Freeman recognizes that soil tests are needed before it will be known whether the floods would do the cut-

ting. Clayey material like that observed by the writer in the San River south of Hungtze Lake would cut very slowly. From general observations, the routes nearer the old Yellow River course (Fig. 22) would be expected to be mostly Yellow River silt with the least admixture of more resistant material from the local mountains.

There are two other obvious difficulties with this method: First, the initial channels are at the toes of the dikes and what is probably the cheapest protection, namely, groynes projecting 100 ft. or more beyond the inner face of the dike, would be impracticable. (It will be noted that this groyne method is that recommended by Mr. Freeman for the Yellow River (Fig. 19)). Second, the floods do not come in orderly sequence. If, after the initial channels were dug, a big flood came first, it would find inadequate channel capacity. The usual sequence of a number of small floods followed by a big one would be more fortunate. A concave bank on the Yellow River resists surprisingly well, although pieces of the bank break off every few hours during a flood. The water of the Huai River would work faster than that of the Yellow River, as it is not so heavily laden with silt. After giving due weight to all unfavorable conditions, it seems probable that considerable help could be had from the flood waters in digging the channel.

Mr. Freeman refers to flood waves in the Wen River, in Shantung. These waves are apparently thought of as akin to bores. The survey of scores of 1918 flood heights along about 37 miles of the Wen River and about 47 miles of the Ssu River, in Shantung, showed surprisingly uniform slopes in long stretches. Mr. Freeman thinks these marks may show merely the height reached by a traveling wave crest. In a measure, this is true and is recognized of flood heights along all rivers. The real question seems to be whether the crest has a visible face approaching a bore or whether it is of such gentle slopes as not to be recognizable as a crest without simultaneous gauge readings at intervals along the river. If there had been a flood wave resembling a bore, it would seem that some of the many natives interviewed would have noted it. In the stretches surveyed the slopes of the rivers are about 1 : 1 100 to 1 : 2 000 for the Wen and 1 : 1 300 to 1 : 3 600 for the Ssu, steep to be sure, but not slopes of torrential streams. A flood of more than one-half the estimated maximum was observed in the Ssu River in 1920. It was caused by 4 to 5 in. of rain beginning at 6:30 P. M., and continuing until 11 A. M., the next day. The river rose about 13 ft. between 5 A. M. and 9 A. M., and remained at the maximum for an hour, after which it fell at the rate of about 8 in. per hour. A wave crest, which takes an hour to pass seems to be rather flat, not like a bore. The writer doubts whether the larger waves were much more abrupt of face than that of 1920. There is serious question, however, as to the accuracy of slope and area methods in estimating flows of even such comparatively flat crests.

The writer spent some months in the study of irrigation problems in Shansi and elsewhere in China. Reservoir sites were found, but everywhere the land was composed of the same loess, or fine silt, and the rivers, most of which are feeders of the Yellow River, carried enormous loads of this loess, even after light showers. Reservoirs would fill up in a few years. A patient

study and gradual planting of vines or special grasses in the myriad gullies, perhaps, might arrest this erosion. It has not been tried and the areas to be treated would be enormous. With conditions as they are, the writer could only admire the simple and efficient use which, in general, the Chinese were making of the muddy flood waters, leading them out at favorable seasons in a network of canals and laterals, which are operated on a co-operative plan with each benefited hamlet contributing to the upkeep that consists largely in cleaning out the silt and tending the openings during the irrigation season. Some improvements appeared to be possible, but he is a bold person who is sure, after a few weeks of observation, that the Chinese have failed to see some easy chance of utilizing natural advantages for the benefits of agriculture. They appear always to have grasped the basic ideas, but to have failed in labor-saving devices. A plethora of cheap labor has doubtless dulled that side of their intelligence.

The hope of the nation is not only in great public works of improvement, important as they are, but in the increase in manufacturing and diversification of industry in the interior, the general introduction of modern comforts, the consequent raising of standards of living, and the limitation of excessive increase in population. Without these latter changes, the population would simply grow to the limit permitted by increased food production. The diversification of industry is progressing in a way to make the outlook hopeful. Increase in the number of highways and railroads is of paramount importance, both in permitting relief of famine which is always comparatively local, and in spreading progress of all kinds.

E. W. Lane,* Assoc. M. Am. Soc. C. E. (by letter).—The papers in the Symposium on Flood Problems are interesting to the writer, largely because of his connection with several projects of that nature. The writer served for nearly four years on the Engineering Staff of the Miami Conservancy District. Since leaving that work, he has been employed by the Morgan Engineering Companies on the Grand Canal, in Kiangsu Province, China, and on bank revetment work on the Yangtze River. He has also made some experiments on the standing wave and the hydraulic jump, and, therefore, will confine his remarks to flood conditions in China, which are not as well known to American engineers, and to the subject of standing waves.

Mr. Freeman has rightly stated that "there is no part of the world where the science and arts of hydraulic engineering could do more for humanity than in China". No engineer can see the great devastation wrought by the floods there without being impressed with the great benefit which would result from well planned engineering works.

The flood problems of China are unusual in many ways and, therefore, may demand an unusual solution. In solving these problems, great injury will be done by a too slavish following of precedent established in other countries. On the other hand, great damage would result from the failure of any large project purporting to represent modern methods, not only on the project itself, but on many others as well, as the extreme conservatism of the

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Chinese makes it difficult, at best, to establish sufficient confidence to allow the construction of works along modern lines.

The plan proposed for the control of the Yellow River, although somewhat radical, seems to be based on sound principles. Some difficulty might be experienced in digging the preliminary channels of sizes sufficient to produce a scouring velocity. The practicability, however, would seem to be largely a matter of the cost of this plan as compared with others which might be proposed.

Before going to China, the writer had outlined in his mind the principles along which one possible solution of the Yellow River problem might be made. It is, however, even more radical than that proposed by Mr. Freeman, and is based on principles which, as far as the writer knows, have never been used before in river work. If a reservoir site of sufficient capacity could be found on the Yellow River, at the head of its delta, in which it could deposit part of its silt, the water, after leaving the reservoir, would again take up a load of sediment from the river bottom and carry it to the sea, thus excavating a great quantity of earth from the river bottom and causing a lowering of the river bed. The practicability of such a scheme depends on favorable topographic conditions and other factors which the writer has been unable to study in detail. The use of this principle should be recommended only after a long period of study and experiment, but it offers the possibility of a cheap solution of the problem and, when a final decision is made on the plan to be adopted, no possibility should be overlooked.

Protection Against the Hwai River Floods.—For the last two years, the writer has been engaged on the improvement of the Grand Canal in Kiangsu Province, which is closely related to the floods of the Hwai River. During this period occurred one of the worst floods in a century, causing not only a great loss of life from the flood itself and the famine following it, but also a property loss of millions of dollars.

The problem of protection from the floods of the Hwai River has been the subject of at least seven reports, each of which recommended a different plan. The writer is of the opinion that the time and study given to the problem thus far has not been sufficient to devise the best plan. Until more surveys are made, and all the plans heretofore proposed and all other possibilities are taken up in detail and compared on an equal basis, it is impossible to select the best.

The plan proposed by Mr. Freeman is one of the most interesting of these possibilities, but it is based on the assumption that the soil along the route of the proposed channel is easily eroded, as is that of the Yellow River. The writer regrets to state that the information which he has been able to collect by observation and discussion with others familiar with the region traversed by the proposed channel, makes it seem unlikely that a route can be found without an undesirably large proportion of the soil which will be difficult to erode. At Point *R*, Fig. 22, one of the alternative channels crosses a large hill of hardpan and near Point *A*, in the construction of a number of ditches for famine relief, considerable clay was found and in one place a layer of clay

and sand was so hard that it had to be broken with hammers. Persons familiar with the region through which the proposed channel runs, state that "from the edge of Hungtze Lake north to the Old Yellow River bed the soil is old lake bottom, and is clay, a brownish color." About 35 miles northeast of the Grand Canal, the proposed channel "comes again to old lake beds and to clay soil. In the Hai Chow Section, the land is part sandy and part a sort of clay, a good deal of gravel in it."

About half way from the Grand Canal to the sea, the proposed route will encounter "a very mean, sticky soil unlike anything I know elsewhere. It gets brick hard under warm sunshine." In examining this plan, a number of questions arise such as, the difficulty of obtaining sufficient slope, hydraulic radius, and smoothness in the preliminary channels sufficient to produce scouring velocities; the interference of this channel with the Grand Canal and the irrigation water supply of the land to the east; the provision for taking care of the existing streams which are crossed by the channel, and the areas of land from which natural drainage is cut off; the danger of silting the main channel by floods coming in from the branches; the cost of the plan as compared with others proposed, and the safety of carrying water at high velocities so near the levees. Sufficient data are not included in Mr. Freeman's paper to make a discussion of these questions advisable.

As a proof of the writer's statement that the problem has not yet received sufficient study to select the best plan, may be cited one of the possible solutions which heretofore has not been considered, but which may be found to be cheaper than any previously proposed.

At present, the Hwai discharge flows to the Yangtze River through the San River, Pao Ying Lake, Kao Yew Lake, Sha Pao Lake, and thence through small channels to the Yangtze. Both the San River and these channels between Sha Pao Lake and the Yangtze have very small cross-sections, in comparison with the quantity of water which they are required to carry, and all pass through comparatively high land. Because of this small cross-section the water surface in these two channels is steep (about 13m. in 270 km.). A great part of the flooding in and up stream from Hungtze Lake is due to the inadequate capacity of the San River, whereas that in the region of Pao Ying and Kao Yew Lakes and the area east of the Grand Canal is due to the small size of the channels leading from Sha Pao Lake to the Yangtze. If the capacity of these channels was sufficiently increased, or auxiliary channels were furnished, the destruction caused by the Hwai flood would be greatly decreased, if not prevented.

The writer believes that, to the east of each of these channels, land could probably be found low enough to construct a flood channel parallel to them, by confining the water between levees, which would be a cheap form of construction. The confirmation of this possibility must await the completion of surveys which are now under way.

The prevention of the floods of the Hwai River is economically justified. The vast areas of shallow lakes which could be reclaimed, would, in themselves, probably pay the entire cost of such work, and the prevention of famine and

loss of crops on those lands now cultivated, but occasionally flooded, would be without cost.

Improvement of the Yangtze River.—The writer agrees with Mr. Freeman that the Yangtze River greatly needs training, but, except in certain cases, where especially valuable properties are threatened, he doubts the economic feasibility of it. That appreciable improvement of navigation by river training could be made, except at great cost, is doubtful, and the Yangtze is already easily navigable for large boats from Shanghai to Hankow, a distance of 600 miles, and ocean-going vessels are able to travel this route during the summer. It is true that the river rapidly erodes the banks in certain localities, but it also builds up the land in others, and the loss in one case is practically balanced by the gain in the other.

Old Chinese Structures and Methods.—The ancient engineers of China built some exceedingly good masonry work. After examining more than fifty sluices and culverts, which are built on small piles, the writer has yet to find one showing evidence of settlement.

Bank protection structures used along the Grand Canal are formed of alternate layers of reeds and mud, and the writer has seen one such structure withstand a velocity of as high as 12 ft. per sec. without injury to the protection, although it finally failed by the scouring of the unprotected earth beneath. Such structures last only three years, but in view of their low cost, as compared with the cost of permanent protection, and the high interest rates prevailing in China, they are more economical than one would conclude at first thought.

The method of closing a break in a levee with these reeds is ingenious. A large boat is anchored a short distance from the edge of the break, on the line of the dike which is to form the closure. From seven to nine bamboo cables, about 1½ ft. apart are stretched from this boat to the bank. On these cables is laid first a layer of reeds and then a layer of earth. Alternate layers of reeds and earth are added until considerable thickness is secured, when the ropes are slacked off, allowing the mass to sink slightly. More layers of reeds and earth are added, and the mass is again allowed to sink, the operation being repeated until it reaches bottom. The ropes to the boat are then extended back to the end of the dike, over the top of the reedwork, and fastened to stakes. The boat is then moved out about 10 ft. and another section built. This method is successful where the bottom is hard enough to withstand scour. Such dams are built every year across the San River and the streams leading from Sha Pao Lake to the Yangtze, in order to keep the water in Hungtze, Pao Ying, and Kao Yew Lakes and the Grand Canal at the depth required for navigation. Although the cost is considerable, it is doubtful whether modern movable dams would be cheaper.

Mr. Freeman's statement is correct that it will often be better to use local Chinese labor for excavation in place of modern machinery. Excavation by hand labor in China is remarkably cheap, and because of the great available supply, particularly in the winter, excavation can usually be rapidly done. During the winter of 1921 a section of the Grand Canal was drained and

excavated, navigation being detoured. In two months of unfavorable weather 560 000 cu. yd. were excavated, at a cost of about 6 cents per cu. yd. At one time, 8 000 men were employed. The excavation was difficult, because of the nature of the soil, but on other work in this vicinity, where conditions were favorable, earth was excavated and carried about 240 ft. at a cost of from 3 to 4 cents per cu. yd. The cost of excavating a small ditch, from which the earth was not carried, was less than 2 cents per cu. yd. These prices are somewhat exceptional, but serve to indicate what may be done. In the work just mentioned, all the earth was carried in baskets, but where long hauls are required, industrial railways, with hand-pushed cars, are being used.

Although excavation in the dry is very cheap, the Chinese have not developed any method of digging in an appreciable depth of water. Where conditions are favorable, excavation by pumping can be done cheaply, otherwise there seems to be, at present, no recourse but to modern machine methods. It may be possible to develop a simple and cheap instrument whereby digging to a depth of 6 or 7 ft., by hand labor, may be done, which, under many conditions, will be even cheaper than with modern dredging equipment.

Standing Waves.—The writer is inclined to disagree somewhat with Mr. Grover's classification of standing waves. In the writer's opinion, the tidal bore and the standing flood wave are, in reality, the same phenomena, the only difference being that the tidal bore advances in the direction in which the bottom slopes upward, whereas the standing flood wave advances in the direction in which the bottom slopes downward. The limiting condition for both would be a bottom with no slope, in which case they would be identical. The flood wave will not form a vertical or nearly vertical front if the water in the stream in which it flows before the wave came, had sufficient depth, and controlled by the same laws, the tide will not form a bore under similar conditions.

Another form of standing wave, not discussed by Mr. Grover, is often seen in water flowing at high velocity, particularly below a dam. It usually has a rounded crest and is stationary, and for lack of a better name it may be called a rounded stationary wave. The writer has measured some of these waves in a small model, but has not arrived at any definite conclusions regarding the laws which govern them. The results of these measurements have been presented to the Society by the writer.* A good photograph of such a wave has also been published† by D. W. Mead, M. Am. Soc. C. E. Although these waves usually have rounded crests, they sometimes break on the crests in the same manner as the whitecaps on water waves. In this form, they are similar in appearance to the hydraulic jump, but are different, being probably controlled by the same laws as those which govern the formation of whitecaps.

* "Experiments on the Flow of Water Through Contractions in an Open Channel", *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1920), p. 1149.

† "Water Power Engineering", First Edition, p. 584.

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Paper No. 1506

TRANSMISSION OF PRESSURE THROUGH SOLIDS AND SOILS AND THE RELATED ENGINEERING PHENOMENA

By George Paaswell,* M. Am. Soc. C. E.

With Discussion by Messrs. J. C. Meem, M. L. Enger, F. N. Menefee, E. G. Haines, and George Paaswell.

Synopsis

Modern research, both analytical and experimental, has demonstrated that soil stress phenomena may be made amenable to ordinary elastic analysis. Using some theorems of elasticity, it is possible to sketch—qualitatively for the present—stress paths and deformations.

The paper shows that, in a restricted sense, the present-day "rule-of-thumb" methods of assumed stress paths hold true. Danger zones which these empiric methods do not indicate, do exist, and these zones may affect the safety of structures sustained by these soils.

The paper covers two types of materials, namely, the true granular, such as the ordinary soils, and the concrete aggregates, such as the rock soils and concrete materials. The formulas yield equations showing the distribution of pressure under foundation piers. Although Boussinesq has given some formulas for such distribution, it is felt that the method given in this paper is simpler to follow and to adapt to soil mechanics.

It is shown that all coherent granular materials, including concrete and rock, have like failure surfaces, indicating general laws of failure, a fact that has been verified experimentally.

If the results of the paper are co-ordinated with the results of some present intensive researches in soil mechanics, quantitative formulas will eventually develop, that will place the so-called soil type in the category of engineering structural materials yielding to mathematical analysis.

The transmission of pressure through solids and soils is an engineering phenomenon of great importance. Until recently, however, its analysis has

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received scant attention. The study of the transmission of lateral earth pressure is an offshoot from the general problem. It has received more attention than the general problem, but, as the writer has pointed out,* its solution, as far as rigor and exactness of formulation are concerned, is comparatively unimportant, and existing data and formulas suffice for a proper design of structures built to sustain lateral soil thrusts. For this reason, the phenomena attendant on lateral pressures, as far as the ordinary granular theories apply, are not a part of the subject of this paper, and restriction is made, generally speaking, to the transmission of vertically applied loadings.

The transmission of pressure through solids, although superficially an easy problem to analyze, is one that involves great mathematical difficulties. For this reason, specific problems are generally not analyzed, a few "rule-of-thumb" methods sufficing to indicate the distribution of stress through the material in question.

The problem of the distribution of stress under a single concentrated load applied to a plane boundary of an otherwise infinitely dimensioned solid has received complete study by Boussinesq. The method of investigation follows the method of solution given in his book on the applications of the potential. Love has given a résumé of the methods in his treatise on the mathematical theory of elasticity. Boussinesq has given general expressions for the extension of the results to various types of surface loadings. The expressions are not simple to use or to interpret, and the devices used in this paper seem to lend themselves better to practical study. Professor Horace Lamb† has given expressions for stress distribution under like types of loadings, but the results involve the use of the Bessel Functions and, therefore, require extensive tables for application.

A fairly rigorous solution of the problem should indicate the correct paths of the stress and strain systems, perhaps only qualitatively, and thus aid in securing a better understanding of what takes place in foundations and possibly secure a working clue to the phenomena attendant on earth pressure and its distribution.

The inclusion of the study of earth pressure phenomena with general elastic solid phenomena is made justifiable through the results of recent research. The element of coherence appears to lend quasi-elastic properties to soil. Although it may seem far-fetched to apply elastic theory to a material possessing finite voids, with consequent large discontinuities, yet experimental data seem to indicate that the coherent soil does acquire sufficient elastic properties to make it act, in some ways, like an elastic solid. Thus‡:

"* * * Practically all soils possess elastic properties, the amount depending on the character and quantity of the cementing material giving it cohesion and the water content. If unequal loads per unit of area are placed on such soil, unequal settlements will take place. The soil, however, will recover on the removal of the loads unless the latter are great enough to over-

* "Effect of Practical Factors on Earth Pressure Theory", *Engineering-News Record*, Vol. 86, p. 682.

† *Proceedings*, London Mathematical Soc., Vol. 34, p. 276.

‡ Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., *Proceedings*, Am. Soc. C. E., August, 1920. Papers and Discussions, p. 909.

come its elastic properties, by breaking down its cohesion. Such action is most apparent at some depth beneath the surface, as the displacement is retarded by the restraining action of the surrounding soil. This unequal settlement, although slight, may be sufficient with certain soils and structures to cause trouble.

"As evidence of such action the rebound, or rising, after releasing the jacks, of piles forced into the soil by direct pressure, may be mentioned. The action is also apparent on releasing jacks and wedges when cribwork is used in temporarily supporting heavy loads, as in the underpinning of buildings, etc."

At the conclusion of a discussion on earth-pressure theories* it was noted: "These facts indicate the existence of relations between stress and strain in sands similar to those which are known to exist for solid bodies."

The writer has previously pointed out† that the distribution of pressure as experimentally determined by M. L. Enger, M. Am. Soc. C. E.,‡ seems to follow almost the identical law of transmission of stress, as determined from the mathematical theory of elasticity. Whether the soils actually possess elastic properties making them amenable to the mathematical theory of elasticity, is not a matter of importance as long as it is conceded that they behave in a manner similar to elastic bodies as far as the transmission of pressure is affected.§

The general theory of the transmission of stress through a solid does not lead to simple solutions of usable value. However, if in place of dealing with a solid of three dimensions, a plane section is taken as typifying the stress and strain condition, it is possible to obtain usable solutions. It must be emphasized that, at present, rigorous results or exact formulas are out of the question. All that is sought is a suggested trace of the transmission of pressure. Fortunately, this specific problem, that of the transmission of pressure through a solid, is substantially solved for the engineer's purpose when qualitative results are indicated.

This paper is confined to the discussion of two cases:

Case I.—The transmission of pressure through a solid or through a coherent soil, under an applied vertical loading.

Case II.—The transmission of pressure through a solid subjected to stresses produced by its own weight.

Utilizing the elastic analogy, as pointed out, application will be made under Case I to the transmission of pressure through soil foundations and under Case II to the general distribution of pressure through earth masses, rock, and other soils.

Case I

The analysis of the infinitesimal displacements and stresses to which an element of a body is subjected when a given force system is applied

* "Old Earth Pressure Theories and New Test Results", by Dr. Charles Terzaghi, *Engineering News-Record*, Vol. 85, p. 632.

† "Retaining Walls: Their Design and Construction", p. 31.

‡ *Engineering Record*, Vol. 73, p. 106.

§ The application to bodies not possessing isotropic properties is indicated by the following remarks of J. H. Michell (*Proceedings*, London Mathematical Soc., Vol. 32, p. 256): "It appears, therefore, that the law of depression is the same as for an isotropic solid; consequently, the applications of the law, which were made by Boussinesq and Hertz, to problems concerning isotropic bodies in contact, may be at once extended to ælotropic solids here considered, with the limitation that the normal to the plane of contact must be an axis of elastic symmetry."

at the surface of the body, leads to the establishment of several differential equations containing the displacements or the stresses as unknowns. For the purposes of this paper, it is not necessary to burden the reader with the development of these equations, which, to those interested, may be found in any standard treatise on elasticity.* The general theory of their solution involves considerable analysis, but with restrictions, as noted subsequently, a solution may be found by following a theorem of Michell. When the stress analysis is confined to the discussion of the stress system in a plane section of the body, it is possible to reduce the general equations of elasticity to a simple form, involving one unknown termed a stress function. The displacements are the differential coefficients of the stress function, and the stresses may be expressed in terms of the displacements. The problem is to find the form of the stress function, which satisfies the differential equation and leads to proper values at the boundary. Michell† has found the form of the stress function for the case of a uniformly distributed loading along a part of the top of the plane (the type of loading covered under Case I).

The general equation of elasticity for a solid, when it is subject only to loadings applied externally, is of the form:

$$\frac{d^4 F}{d x^4} + \frac{2 d^4 F}{d x^2 d y^2} + \frac{d^4 F}{d y^4} = 0$$

where F is known as the stress function. The stresses are found from the solution of this equation and lead finally (expressed in polar co-ordinates) to the form:

$$P = \frac{1}{r^2}\frac{d^2 F}{d \theta^2} + \frac{1}{r}\frac{d F}{d r}; \; Q = \frac{d^2 F}{d r^2}; \; U = -\frac{d}{d r}\left(\frac{1}{r}\frac{d F}{d \theta}\right)$$

A form of the stress function may be found by inspection. Referring to Fig. 1, the point, O, is referenced to each of the terminals of the loading, A and B, by a separate co-ordinate system. If the stress function is assumed to be of the form,

$$F = r_1^2 \, \theta_1 - r_2^2 \, \theta_2$$

the stresses may be assumed as being made up of two parts: The derivatives with respect to the origin at A giving one set of partial stress values, and the derivatives with respect to the co-ordinates of the other terminal giving another partial set; the sum of the two sets giving the total stress value at the point, O. Thus, with respect to the origin at A:

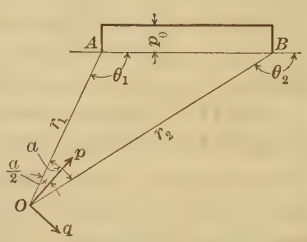

Fig. 1.

$$P = 2 \, \theta_1; \; Q = 2 \, \theta_1; \; U = -1$$

and with respect to the origin at B:

$$P' = -2 \, \theta_2; \; Q' = -2 \, \theta_2; \; U' = +1$$

* "A Mathematical Treatise on the Theory of Elasticity", by A. E. H. Love; or in the Appendix of "The Elasticity and Resistance of Materials", by W. H. Burr.

† *Proceedings*, London Mathematical Soc., Vols. 31, 32, and 34. For the theorem established on the following pages see especially Vol. 34.

To find the principal stresses at the given point, O, proceed as follows: Assume that the present axes are turned through the angle, ϕ. By the standard theorems for the transformation of stress axes* the stresses, P'', Q'', and U'' (these stresses are normal to the new axes), are related to the stresses referred to the original axes, as follows:

$$P'' = l_1^2\, P + m_1^2\, Q + 2\, l_1\, m_1\, U$$
$$Q'' = l_2^2\, P + m_2^2\, Q + 2\, l_2\, m_2\, U$$
$$U'' = l_1\, l_2\, P + m_1\, m_2\, Q + (l_1\, m_2 + l_2\, m_1)\, U$$

where l and m represent the direction cosines (the cosine of the angle that a line makes with a given axis is termed a direction cosine of that line) of the new axes with respect to the old. The direction cosines have the following values:

$$l_1 = -\, m_2 = \sin \phi;$$
$$l_2 = \quad m_1 = \cos \phi.$$

Introducing these values in the transformation equations, previously mentioned, and combining the equations by addition and subtraction, the following relations are found:

$$P'' + Q'' = P + Q$$
$$P'' - Q'' = -\,(P - Q) \cos 2\,\phi + 2\, U \sin 2\,\phi$$
$$2\, U'' = \quad (P - Q) \sin 2\,\phi + 2\, U \cos 2\,\phi$$

These equations may be reduced to a much simpler form if the equation,

$$e^{i\phi} = \cos \phi + i \sin \phi$$

is borne in mind. i has the usual significance as the square root of -1 and e is the exponential base. With this relation, the transformation equations may be expressed as,

$$P'' - Q'' - 2\, i\, U'' = e^{2i\phi}\,(Q - P - 2\, i\, U)$$

These are the general equations of a stress transformation. Now, give the axis of transformation a definite turn so that the axes of stress are the bisectors of the angle, a, as shown in Fig. 1. The stress components are again separated into two parts, referring, in turn, to either origin, A or B.

For Origin A, the axes may be considered to have been turned the amount, $\frac{a}{2}$, and for Origin B, the axes may be considered to have been turned the amount, $-\frac{a}{2}$. Denote the total stresses referred to the new axes as p, q, and u, respectively. These stresses are then compounded of the two transformed stress systems, as previously found, substituting, respectively, $+\frac{a}{2}$ and $-\frac{a}{2}$, for the angle, ϕ. Two equations are then found, as follows:

$$p + q = P + Q + P' + Q' = 4\,(\theta_1 - \theta_2) = -\,4\,a$$
$$p - q - 2\, i\, u = e^{ia}\,(Q - P - 2\, i\, U) + e^{-ia}\,(Q' - P' - 2\, i\, U')$$
$$= 2\, i\, (e^{ia} - e^{-ia})$$

* "A Treatise on the Mathematical Theory of Elasticity", by A. E. H. Love, 1st Edition, p. 62.

or, since,

$$\sin a = \frac{1}{2}\,(e^{ia} - e^{-ia})\,i$$

the expression becomes equal to $-4 \sin a$, and, finally, u is zero, showing that only normal stresses exist on the axes and that these are, therefore, the principal stresses at the point, O. The principal stresses have the values:

$$p = -2\,(a + \sin a)$$

$$q = -2\,(a - \sin a)$$

It is noted that when the points are on the plane boundary, the principal stress reduces to a normal pressure of 2π, and account must be taken of this relation in reducing the forms to take care of any value of the surface loading.

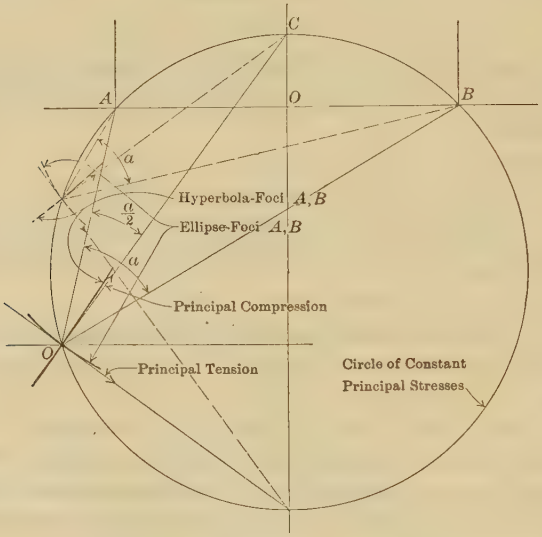

FIG. 2.

In the general analysis, the selection of the signs does not affect the validity of the result, and as it is apparent by inspection that the principal intensities must be of opposite sign, the character of the principal stress has been chosen with the signs as noted subsequently. To reduce the equations to a form satisfying a uniform loading of p_0 per unit of length between the points, A and B, a simple substitution gives the following forms for the principal stresses:

$$p = \frac{p_0}{\pi}\,(a + \sin a)$$

$$q = -\frac{p_0}{\pi}\,(a - \sin a)$$

From these simple relations, a number of important corollaries are found. Referring to Fig. 2: A circle passing through the point, O, and the termini

of the load line, A and B, is the locus of constant values of the angle, a, that is, every point on this line has the same value of the angle, a. Therefore, this circle is the locus of equal values of the principal stresses, p and q. From the properties of the circle, it is seen that the principal stresses are directed toward the upper and the lower extremities of the vertical diameter.

Conics which have the same foci are termed confocal conics. From the properties already noted, it is seen (the proof is not difficult) that, at any point in the body, the principal stresses are found in direction by the intersection of two confocal conics passing through the point, the foci being the termini of the loads, A and B. The principal compressive stresses are along ellipses and the principal tension stresses along hyperbolas. Confocal conics always intersect orthogonally, that is, at right angles to each other. In a material weak in tension, the curves of possible failure are the hyperbolas, and the tension stresses are normal to these curves. That the surfaces of failure are of this character in deep rock cuts has been noted by H. G. Moulton,* M. Am. Soc. C. E., and also in illustrations of the paper of Professor Enger, previously mentioned, in which the stress paths are seen to follow along curves hyperbolic in shape. (See, especially, the movement under an 8-in. block.)

From the intersection of the circles of uniform stress with the confocal conics, the rate of change of the principal stresses along each conic may be noted. (See Fig. 3.)

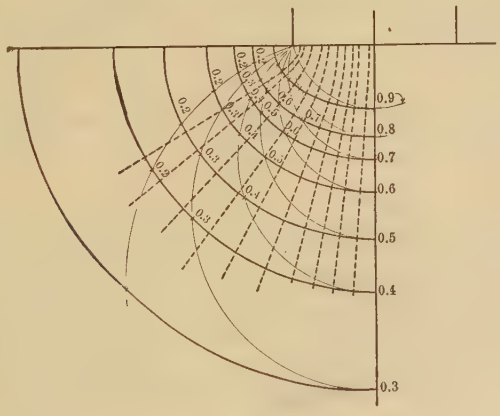

FIG. 3.

It is probably of most importance to note the distribution of pressure along the horizontal planes. As given, the equations are not in form for ready use, and it is necessary to resolve them into the vertical and horizontal components. (See Fig. 4.)

Let ϕ be the angle that the principal compression stress, p, makes with the horizontal. Then, the vertical component is found from the equation:

$$p_y = p \sin \phi - q \cos \phi$$

* "Earth and Rock Pressures", *Transactions*, Am. Inst. of Mining and Metallurgical Engrs., Vol. 63 (1920), p. 327.

For simplicity of expression, and as only comparative studies are required, it will be assumed that the intensity of loading along the surface is unity, and that the loaded length extends unit distance either side of the y-axis. Using the values of p and q, as previously found, with the value of the load as unity, the last expression takes the form:

$$p_y = \frac{a\,(\sin\phi - \cos\phi) + \sin a\,(\sin\phi + \cos\phi)}{\pi}$$

From Fig. 4, remembering that c is 1,

$$\tan\phi = \frac{y + \tan\frac{a}{2}}{x}$$

Substituting this value in the latter expression, there is found:

$$p_y = \frac{a\left(y + \tan\frac{a}{2} - x\right) + \sin a\left(y + \tan\frac{a}{2} + x\right)}{\pi\sqrt{x^2 + \left(y + \tan\frac{a}{2}\right)^2}}$$

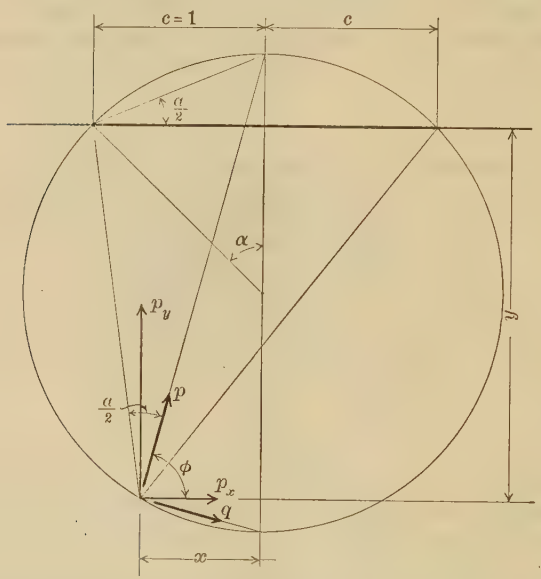

FIG. 4.

To obtain the relation between the angle, a, and the co-ordinates, x and y, let R be the radius of the circle. Then,

$$R^2 = x^2 + (y - \cot a)^2$$

and, as c is 1,

$$R^2 = 1 + \cot^2 a$$

and a is found from the equation,

$$\cot a = \frac{x^2 + y^2 - 1}{2y}$$

Fig. 5 illustrates the distribution of pressure at the various horizontal planes when the surface is loaded with a unit intensity per unit length, one unit length being loaded at the side of the y-axis. The intensities of pressure are in decimals of a unit and to apply to any actual condition of loading, the skeleton intensity diagram is multiplied by the actual surface intensity of loading.

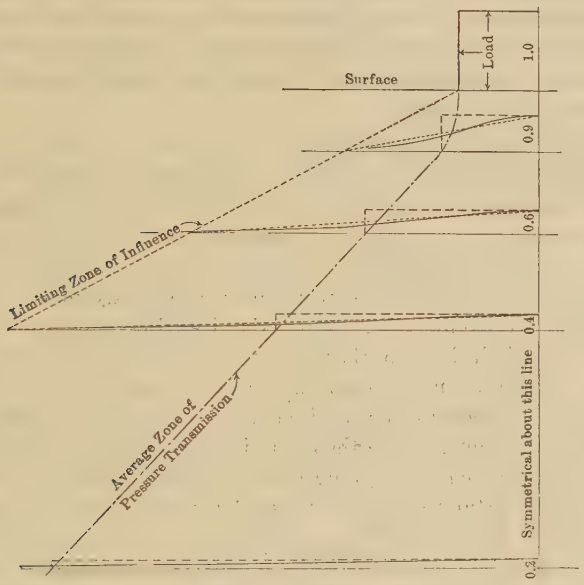

FIG. 5.

Before analyzing these results, a review of existing methods may be presented. Ordinary practice assumes a uniform distribution of pressure along any horizontal plane below the loaded surface, increasing in spread as the distance below the surface increases and confined, roughly, within planes making an angle of 45° with the vertical; that is, pressure is assumed to distribute itself along a 1 : 1 slope. Lacher* has given the following expression for the vertical live load intensity at any depth, h, below the surface (due to locomotive wheel loads):

$$p = \frac{11\,000}{(8 + 2\,h\,x)}$$

where x is the inclination of the spread planes, in fractions of a foot per foot of depth. Recently, experimental data have illustrated in better fashion the distribution of stress through a foundation. The Progress Reports of the Special Committees on the Bearing Value of Soils for Foundations and on Stresses in Railroad Track, of the Society, have shown graphically the actual loading conditions, as determined by the Goldbeck strain gauge. It is clear

* *Journal*, Western Soc. of Engrs., Vol. 20.

that data in general on such stress distribution are meager and that engineering design has followed traditionary measures for such computations as were deemed to be necessary, rather than to attempt methods of rational analysis.

Referring to Fig. 5, it may be noted that the stress distribution across any horizontal plane is not uniform, but has a maximum at the center and diminishes in intensity as the distance from the central axis increases. This has been demonstrated by the other theorems of elasticity; the "simple solution" method of Boussinesq, to which reference has already been made, leads to a similar distribution of stress. The strain-gauge readings of the experiments of the Special Committees of the Society, previously referred to, show the same characteristic loading.

Averaging the intensities of load on the several planes, it is seen that a 1:1 slope gives a reasonable distribution of pressure, if the effects of unequal loading are ignored. To this extent, the "rule-of-thumb" method previously quoted is correct. It is noted, however, that the limiting zone of pressure influence is contained within a 1:2 slope. The formulas, as given, involve no elastic constants and with the reservation previously made, the results may be extended to coherent soils. With the limiting zone of surface-pressure influence in mind, it is seen that the usual criterion of a 1:1 slope is not always safe to follow in determining loads on sub-surface structures, or in the question of underpinning and shoring structures adjacent to or contained within such zones. The theorems just established and the figures based on them are predicated on a plane extending indefinitely in both directions. If the horizontal distances are finite, as they usually are in engineering structures, the curves will probably be modified. It is recognized, however, that such modification will not materially affect the qualitative results already given and, within reasonable distances away from the applied loading, the zones of influence are shown with a fair degree of accuracy.

A study of the expression for q, the principal tension, shows that for distances close to the origin and to the surface of loading, a is equal to 180° and as sin a is zero, the tensile stresses are a maximum, and a tendency to split along the hyperbolas should be found. The material adjacent to the applied loading and outside the zone of influence has an inhibiting effect on the tension, but this restraining effect decreases as the distance from the surface increases. Thus, although the maximum tension stress is indicated close to the surface, the uncompensated stresses farther away, along the hyperbolas, may produce dangerous stresses.

The following is quoted from the Progress Report* of the Special Committee on the Bearing Value of Soils for Foundations, etc.:

"* * * In the case of an elastic solid the outer region of pressure and tension contributes to the distribution of stress, while in the case of soils, which cannot support tension in any marked degree, the surface between these regions presents a surface of discontinuity of strain resulting in a greater concentration of stress within the compression cone. This cone, of course, will have different angles for different types of materials, but it is probable that it will not differ widely in ordinary soils."

* *Proceedings*, Am. Soc. C. E., August, 1920, Papers and Discussions, p. 933.

The "cone" of compression, as noted in the report of the Special Committee, may be compared to the zone of the transmission of average pressure, with which it roughly coincides.*

Further reference may be made to the Progress Report of the Special Committee on Stresses in Railroad Track,† in which a study is had of the transmission of pressure through granular materials. There is a general qualitative agreement in the manner of the distribution of the pressure along horizontal planes, and it is probable that if an equation for the location of equal vertical loads was derived, it would be found that the coherent and the elastic solids have the same type of "bulb" curves as the granular solids.

The agreement in stress distribution between material of so varied a character as that covered in the various reports mentioned, as well as the elastic solids discussed in this paper, seem to indicate that stress distribution under a surface loading follows a simple general law of distribution, regardless of the character of the material.

As long as emphasis is placed on the qualitative worth only of the results of the theorem of Michell, the previously mentioned conclusions may prove of service. It seems hardly necessary to point out that the actual stress values cannot possess great validity the further the material under discussion departs from the isotropic properties of the solids on which the theory of elasticity has been predicated. Qualitatively, the 1:1 slope has been shown to be a reasonable assumption for the distribution of stress through a mass, although the maximum intensity of pressure is greater than the average, and soil conditions may be such that this latter condition will control the proper layout of the footing. The fact that the ground may heave adjoining a heavy concentration on fairly soft bottoms, may tend to dissipate some of the inequality of the loading, but the distribution, as indicated in Fig. 5, is worthy of careful study.

Case II

Referring to Fig. 6, there is a typical case of a block of material of any character subjected to the forces produced by its own weight. The dimensions of the block may be assumed to be large, so that application may be made to deep rock cuts and to the action of coherent earths. Again, the quasi-elastic properties of such soils may be utilized in developing the equations for the distortions which the mass suffers under the body forces.

The analysis of this material is comparatively simple, following the analysis of a similar problem by Love,‡ who has analyzed the case of a hanging body suspended to an infinite solid. The present problem is merely the converse of this case.

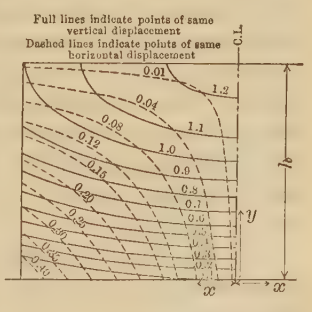

Fig. 6.

* In the text by Love, previously mentioned, the surface of discontinuity between displacements from and to the central axis is also a cone, the angle of divergence of which, for an isotropic solid, is roughly 68°, that is, the sides of the triangle make an angle of about 32° with the vertical.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1409.

‡ "A Treatise on the Elastic Theory of Elasticity", 2d Edition, pp. 124–125.

The only force acting on any element is the weight of the material above it. If the y-axis is measured from the surface and the unit weight of the material is w, the vertical force on the elementary prism is $w\,y$. As the extension is proportional to the force, the extension in the vertical direction becomes $\frac{w\,y}{E}$, in which E is the modulus of extension for the material. If the ratio of lateral to vertical extension is termed K (the usual Poisson ratio), then the lateral extension is $\frac{K\,w\,y}{E}$. These two relations are the foundation equations on which the general stress equations are based, and the differential equations of elasticity may be solved to satisfy these equations, noting, also, that there is no force system external to the body. If the vertical displacement is u and the horizontal displacement is v, the solution of the differential equations leads to the following values:

$$u = \frac{K}{E}\,w\,x\,y$$

$$v = \frac{1}{2}\,\frac{w}{E}\,(h^2 - y^2 - K\,x^2)$$

Before applying these expressions, it may be well to consider what the basic assumptions are and what limitations must be placed on the interpretations of the results. The differential equations are derived fundamentally from the linear stress-strain law, that is, Hooke's law. The material must be isotropic, that is, it must possess uniform elastic properties in the directions of the principal axes, or it must have elastic properties symmetrical about the y-axis. A homogeneous rock mass may fulfill these conditions to some extent. A coherent soil may act as if it, likewise, possesses such elastic properties. It should be clear, however, that any geologic formation can only be treated sketchily as an elastic solid and the results based on elastic research can be most precursorily applied. Experimental observations seem to place more optimism on the application of the elastic theory and, surely, for mere qualitative studies, the elastic theory will provide good working clues if nothing else. Thus, a mere sketch study of the stress systems will indicate weak planes of resistance and also any incipient motion of the mass, and it is in this sense, and probably in this sense only, that the analysis can be made to serve practical purposes. With this point in mind, the mathematical work may be continued.

The previously derived expressions for u and v determine the movement of the particles of the body under stress and, for the ordinary elastic solids, actual stresses may be derived from these displacement equations. These stress equations will not be derived for the material at hand. Plot two families of curves: (1) The curves formed by placing u equal to a series of constants; and (2) those formed by placing v equal to a series of constants. A system of ellipses will then indicate the points of the same downward motion

and a system of equilateral hyperbolas will indicate points of the same lateral movement. (See Fig. 6.) It is seen that the maximum lateral movement occurs at the bottom of a cut, giving a theoretical verification of the fact, noted in practice, that coherent earths, such as clays and soft rocks, tend to push out at the bottom. The ratio of u to v gives the direction at each point of the impending motion, and Fig. 7 illustrates such lines of actual particle paths. This, again, seems to confirm observations made of the action of materials in deep cuts* and indicates a possible mode of failure of retained banks, or of braced banks, if the sheeting gives way. Geological formations appear to follow a law of the type indicated, as far as the formation of gorges, etc., is affected.

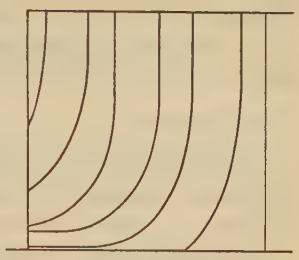

FIG. 7.

In Fig. 6 and Fig. 7, values corresponding to a concrete have been used in plotting the curves. Thus, w is taken at 150 lb. per cu. ft.; E at 1 500 000 lb. per sq. in.; and K as one-fifth.

One hesitates to draw any further conclusions from this mathematical work, except the qualitative ones just observed, although the partial concordance with experimental data may tempt one to carry the analysis a trifle farther. The motion of the mass under stress takes the character shown in Fig. 7 and in coherent soils one would expect that, as failure occurs, that is, as a slip starts, a mass would break away with a bounding surface, as shown by one of the curves in the diagram. Again, there is experimental confirmation of this indicated movement, as it is readily observed that when a coherent bank breaks, the break is roughly hyperbolic in form. The writer has analyzed this point on another assumption,† arriving at about the same conclusion, as far as the form of the surface of slip is affected. The extension of lateral earth-pressure analysis to include such breaking surfaces as a basis of the so-called granular theories may produce results that will bring the formulas in better accord with experimental data. In passing, it may be stated that the familiar theories of Coulomb, Rankine, etc., are predicated on plane surfaces of rupture.

One other method of obtaining a solution of the transmission of pressure through coherent soils and rocks remains to be discussed. It has been suggested that the solution of the problem of the movements of materials in deep cuts may be found by a study of the laws of plasticity and partial viscosity. This method has been used in observing the slides at the Panama Canal,‡ and the basic method is predicated on a plastic condition similar to a hydrostatic condition in which, under a high stress, the materials tend to flow. Dr. Becker has assumed that the weakest resistance to failure in a rock material is that to tangential shear. Under a high pressure, the unit tangential shear

* See the paper by Moulton previously mentioned; see, also, "Mechanics of Panama Canal Slides", by George F. Becker, U. S. Geological Survey.

† *Engineering News-Record*, Vol. 86, p. 682.

‡ "Mechanics of Panama Canal Slides", by George F. Becker, U. S. Geological Survey.

may be taken the same as the unit compression. Thus, if an arc of the surface of failure is analyzed, of length, $d\,s$, and of angular turn, $d\,\phi$, and if the depth is y, so that the unit compression is $w\,y$, as before, and T is the tangential shear that is to cause failure, the following infinitesimal relation holds true:

$$\frac{T}{w} = y\,\frac{d\,s}{d\,\phi}$$

It will be noted, by those interested, that this is the identical equation of the elastica, the type of curve that the slender column takes under an axial load, and the solution leads to the same form of equations.

As the radius of curvature is found from the relation, $R\,d\,\phi = d\,s$, the previously mentioned equation may be written, after placing $\frac{T}{w} = b^2$,

$$R\,y = b^2$$

If the radius of curvature is expressed in the usual form, as a function of the differential coefficients, the following differential equation is formed:

$$-\frac{\left[1+\left(\frac{d\,x}{d\,y}\right)^2\right]^{\frac{3}{2}}}{\frac{d^2\,x}{d\,y^2}}\,y = b^2$$

Replace $\frac{d\,x}{d\,y}$ by p, and there is found,

$$-\frac{(1+p^2)^{\frac{3}{2}}}{\frac{d\,p}{d\,y}}\,y = b^2$$

noting that the second derivative is expressed in terms of p by the following relation,

$$\frac{d^2\,x}{d\,y^2} = \frac{d\,p}{d\,y}$$

Separating the variables, the equation becomes,

$$\frac{y}{b^2}\,d\,y + \frac{d\,p}{(1+p^2)^{\frac{3}{2}}} = 0$$

the substitution of tan u for p leads to a simple integration, and the equation becomes,

$$\frac{y^2}{2\,b^2} + \frac{p}{\sqrt{(1+p^2)}} = c$$

If the constant is taken as equal to zero* (making the curve normal to the upper bounding surface), and the equation is then solved for p, or, for its equivalent, $\frac{d\,x}{d\,y}$, there is found,

$$\frac{d\,x}{d\,y} = \frac{u^2}{\sqrt{(1-u^4)}}$$

u is equal to $\sqrt{\left(\frac{y^2}{2\,b^2}\right)}$. The constant, $2\,b^2$, is a large quantity, as a study of

* A study of the curves shown in Fig. 8, will indicate that other values assigned to the constant will merely change the initial direction of the curve a small amount.

the ratio, $\frac{T}{w}$, indicates and, therefore, u will generally be less than 1. A study of the differential equation shows that $\frac{dx}{dy}$ becomes infinite when $u = 1$, and the tangent to the curve becomes parallel to the x-axis. For large values of y, that is, for values where failure is impending, the curve is normal to the vertical, agreeing with the results shown previously. To solve the differential equation, place,

$$\sin \phi = \sqrt{(1 - u^2)}$$

This reduces the equation to the standard form of the elliptic integrals of the first and second kind, with the modulus, k, equal to $\sin^{-1} 45$ degrees. The solution of the equation leads to the following form:

$$x^* = C' + \frac{F(\cos^{-1} u) - 2E(\cos^{-1} u)}{\sqrt{2}}$$

F is the elliptic integral of the first kind and E is the elliptic integral of the second kind.†

For different initial values of x, the curves of incipient failure are shifted laterally along the x-axis, so that the constant merely determines the starting point of the curve for $u = 0$. Fig. 8 shows a family of these curves. When proper values have been found for the material coefficient, $\frac{T}{w}$, these curves may obtain quantitative worth, but as they are shown, they indicate the possible failure surfaces and bring the plastic or after-elastic effect into harmony with the elastic effects shown.

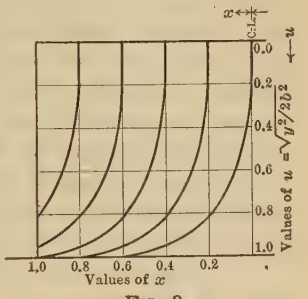

FIG. 8.

It is more than a coincidence that a study of the transmission of pressure by several elastic methods and by an "after-elastic" method leads to failure curves of like form, and the writer feels confident that, as a conclusion, it may be pointed out that the lines of transmitted pressure are substantially the same for all types of material and that the diagrams in this paper represent with qualitative, and, to some extent, quantitative, accuracy, the stress paths in monolithic masses subjected to their own or to imposed weights.

* For simplicity, $b\sqrt{2}$ is taken as unity.

† Tables of their values may be found in Peirce's, "A Short Table of Integrals", and in Jahnkes' and Emdes' Handbook.

DISCUSSION

J. C. Meem,* M. Am. Soc. C. E. (by letter).—The writer agrees with the author in that "the transmission of pressure through solids and soils is an engineering phenomenon of great importance." It appears that discussion of these phenomena has not progressed as rapidly along practical lines as along theoretical ones. The writer has been hopeful that eventually the practical and theoretical may be co-ordinated, resulting in formulas of practical value not only to the designer, but to the constructor as well.

On the question of elasticity of soils, the writer wishes to point out that too much importance should not be attached to the apparent value of this factor, as quoted by the author in connection with the rebound of piles which have been driven by jacks, and the rebound of timbers which have been heavily stressed against soil, when the stress is released; and also what may be called the hydraulic rebound of water-bearing or plastic soils should not be confused with elasticity.

Although in common with all other solids, soil possesses the property of elasticity, the writer does not believe its modulus is sufficiently large to make it a considerable factor under ordinary conditions. Thus, it should be noted that the rebound of a pile which has been forced into the ground by the pressure of a jack is due partly to the recovery from distortion within the elastic limit and the elasticity of the pile itself, rather than to the elasticity of the ground. For instance, in jacking a 20-ft. pile into the ground, the soil below the pile is not stressed perhaps to more than 3 or 4 ft. (from observations on "bulb of pressure" made at Pennsylvania State College), whereas the pile itself is not only heavily stressed within the elastic limit and subject to rebound, but must necessarily be more or less distorted within the elastic limit and, therefore, this rebound may be greatly increased thereby. In jacking sectional shields horizontally into a tunnel heading, the writer has frequently seen the progress stopped by a small stone in front of the nose, which, when removed, caused the section to jump forward perhaps $\frac{1}{8}$ in., due to the elasticity of the blocking behind the jack.

The writer had hoped that the author would concern himself somewhat more fully with the practical problem of earth pressures due to sliding and its related phenomena. There seems to be a disposition among engineers to avoid the use of the angle or plane of repose in connection with earth pressures, but undoubtedly in all soils this plane exists whether it is a straight or a crooked line, or whether the angle it makes with the horizontal is flat or steep. When solid rock or hard clay is tunneled or cut within the limits of its cohesive or sustaining value, the question of the angle of repose does not enter into the problem any more than it would in the case of a hole bored in an ordinary plank. However, when the cut becomes so deep or the tunnel so wide that the rock or clay is no longer able to sustain itself, it breaks down, or tends to do so; and this tendency to slide or break down undoubtedly causes pressure on the wall or the tunnel. Whether this "plane of repose" is called "break-line", or by any other name, some portion of all the soil above this plane

* Vice-Pres. and Chf. Engr., Frederick L. Cranford, Inc., Brooklyn, N. Y.

tends to cause pressure against the vertical face and some portion of all the soil below this plane tends to cause pressure on the tunnel or structure beneath. The writer believes that whether the soil is sand, gravel, boulders, or solid rock tending to break down under sufficiently excessive operations (which, therefore, may cause it to be classed as large aggregates or boulders), the relative pressure on a large scale varies uniformly within reasonable limits, although, on a smaller scale, it may appear to vary within somewhat wider limits.

Assuming then, an angle of repose for normally dry soils, the writer has always assumed arbitrarily that a plane (which he calls the plane of rupture) half way between the vertical and the plane of repose divides this area into two parts, one of which resolved into horizontal components tends to stress the vertical face adjacent, and the other resolved into vertical components rests on the ground beneath the plane of repose. If the soil in the area between the vertical and the plane of rupture is a solid wedge, for instance of clay, a single brace tending to retain this wedge in position should be placed at a point two-thirds of the vertical height; whereas, if this wedge is composed of aggregates tending to slide, there will be a considerable modification of pressure the center of pressure of which the writer has estimated roughly as at a point 0.54 above the bottom or a slight distance above the middle of the vertical.

If, instead of a vertical face alongside, a tunnel structure is placed beneath the soil, then, under the theory advanced, all the material below the two planes of rupture intersecting above the center of the tunnel should bring stress on the tunnel roof, whereas all the soil above these planes of rupture undoubtedly tends to support itself under the arching stress caused by the lateral pressure as long as the integrity of this so-called arch is maintained by the support of the soil below these intersected planes of rupture. It is not probable, however, that all this material below these intersected planes tends to stress the tunnel roof, because some of it also must be relieved by the same arching tendency noted. For safety and because of the lack of experimental data on a large scale, however, the writer has always assumed that the tunnel sustains the weight of all the soil below the intersected planes of rupture, providing the operations are on a scale sufficiently large to cause the soil, whether sand, gravel, clay, boulders, or solid rock, to tend to break down under such operations.

The writer is hopeful that it is not too late for the author to devote some discussion to this phase of his problem.

M. L. Enger,* M. Am. Soc. C. E. (by letter).—This paper is a valuable and stimulating contribution to the literature on the transmission of pressure through soils and solids. The author has been careful to explain that the quantitative results must be expected to be unreliable when applied to ordinary soils. It is surprising that the results of a mathematical investigation of the transmission of stress through an isotropic solid should even approximate the results of experiments on granular materials. As a matter of fact, the resemblance is quite close.

* Prof. of Mechanics and Hydraulics, Univ. of Illinois, Urbana, Ill.

The author has developed a simple method for determining the direction and magnitude of the principal stresses at any point in the mass. In the analysis, the following assumptions were made: (1) That the load was applied to an isotropic solid of infinite dimensions; (2) that the length of the loaded area is infinitely long; (3) that the intensity of pressure on the loaded area is the same at all points; and (4) that the applied pressure is normal to the surface at all points.

The following difficulties appear when it is attempted to apply the results of the analysis to granular material: (*a*) Granular material is not isotropic, (*b*) the principal stresses in granular material are compressive, whereas the author's formulas show one of the principal stresses to be tensile; (*c*) the intensity of applied pressure on the loaded area is not generally uniform; and (*d*), the directions of the applied pressures on different parts of the loaded area are not usually parallel. The variation of intensity of pressure over the loaded area, and the inclination of the pressure at different parts of the area, have been discussed in the Second Progress Report of the Special Committee on Stresses in Railroad Track.*

Assume that the load is applied centrally and vertically on an unyielding plate which rests on granular material of uniform compactness; also, assume that the load is increased gradually from the zero value. The plate will then be depressed slowly, and, at first, the intensity of the pressure of the granular material against it will tend to be uniform, and the direction of the pressures will tend to be normal to the bottom of the plate. As the load is increased, however, there are re-adjustments in the positions of the grains within the granular mass, which tend to modify the distribution of pressure. Such re-adjustments occur most easily immediately below the edge of the plate, because there is less restraint there. As the load increases, the granular material begins to flow from beneath the edge of the plate, decreasing the intensity of pressure near the edge and correspondingly increasing the pressure near the center. The tendency of the material to flow away from beneath the plate is resisted by friction on the plate. The resultant of the frictional force and the vertical pressure at any point makes an angle with the normal, the magnitude of the angle depending on the amount of friction developed. When the applied load becomes great enough, the friction between the plate and the grains, or between the grains, will reach a limiting value beyond which the granular material below the plate will suddenly give away, allowing the plate to drop to a depth at which equilibrium is again possible. Fig. 9 shows a "time exposure" indicating the lines of movement of granular material under conditions which have just been described.

The effect of compactness of the granular material on the carrying capacity and on the distribution of pressure has not been fully recognized. In order that experiments may be analyzed and compared, an index of compactness must be devised. The difference in the behavior of track on new, loose ballast and on well compacted ballast is an illustration of the effect of compactness. Experiments on the transmission of pressure through granular material have usually been made with loose material, and the compacting which it has

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1409.

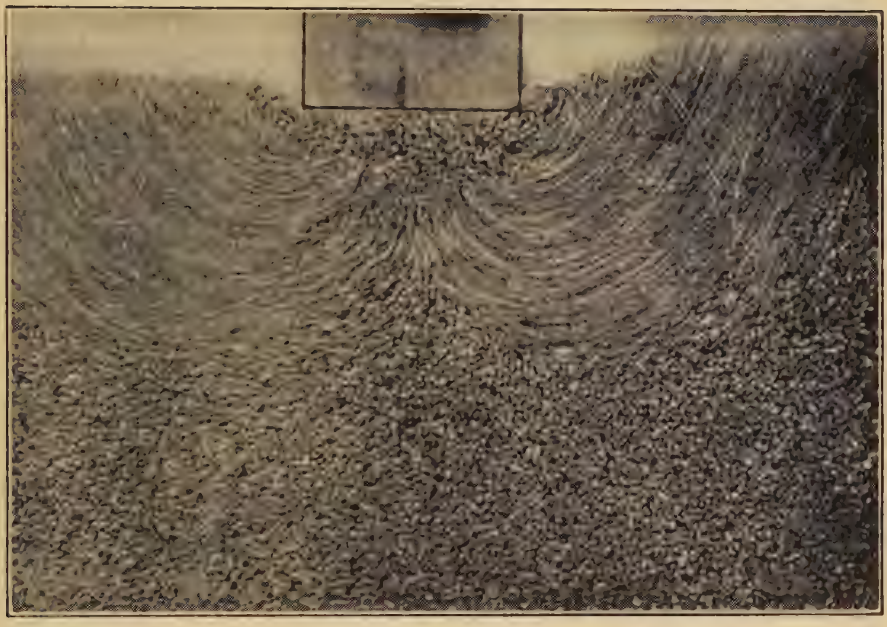

Fig. 9.—Lines of Movements of Ballast Which Has Been Overloaded.

received has been due principally to the loads applied during the progress of the experiments. The result has been that the material directly below the loaded area has been compacted, but the material outside this region remains loose. In the case of railroad track, for example, the jarring action of train loads, the effect of rain, and of tamping, act to compact all the ballast.

Some preliminary experimental work by the writer to determine the effect of compactness on the capacity of granular material for carrying load has given interesting results. Sand was put loosely into a cylindrical measure having a capacity of 0.2 cu. ft. and was compacted by being given a number of ½-in. drops on a flow table. The measure was then weighed after enough sand had been added to fill it. From the weight and the specific gravity of the sand grains, the voids were computed. The measure was next placed on an indicating platform scale and a circular steel disk, 2.5 in. in diameter, was placed centrally on the surface of the sand. Load was applied to the disk until the "yield point" was reached, the load being weighed by the scales. The "yield point" is very marked in the case of compact sand, the disk dropping suddenly ½ in. or more, accompanied by a sound which suggests the breaking of a crust. After completing a test, the sand was emptied from the measure and the operation repeated as described. Different degrees of compactness were obtained by changing the number of drops on the flow table, or by compacting by tamping and pounding.

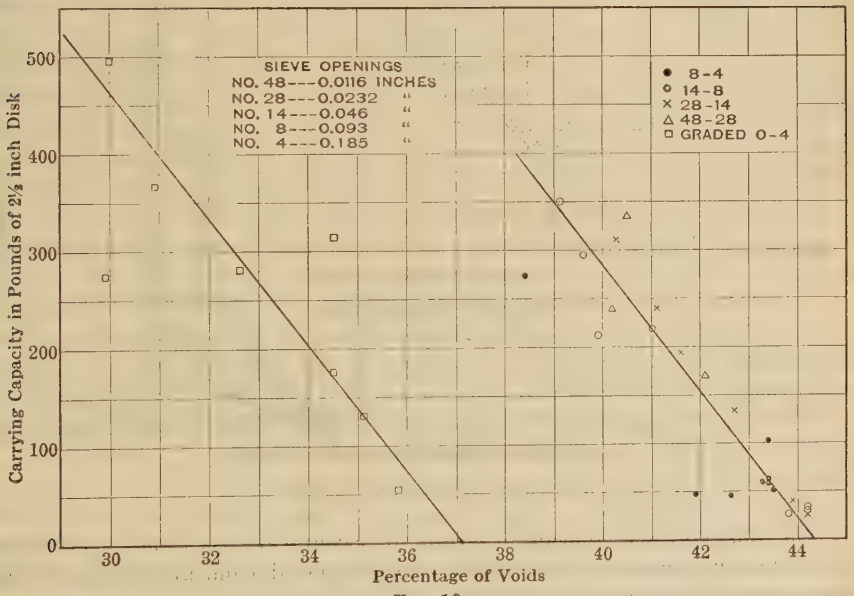

FIG. 10.

Experiments were made on a series of screened sands the largest grains of which were twice the diameter of the smallest grains. The experiments seem to show that the carrying capacity of such a series of sands depends only on the percentage of voids, and does not depend much on the size of the grains. Fig. 10 shows the relation of the carrying capacity of the sands to the per-

centage of voids in the case of the special sands and also in the case of a graded sand. It is unsafe to draw conclusions from a few experiments, but it seems that there is a definite relationship between the carrying capacity and the percentage of voids for any particular sand. It is expected that a considerable program of experimental work along this line will be conducted during 1923.

Granular material lacks elasticity, therefore, Hooke's law cannot apply. Fig. 11 shows the load deformation curves in the case of a railroad tie resting on loose sand. The great depression during the first application of load is clearly shown. It will be noted that when a part of the load is removed there is an actual increase in the depression instead of an elastic recovery. The recovery shown as the last of the load is removed is, in part, due to the flexure of the tie. The load-depression curves shown in Fig. 12 are characteristic of granular material and even of clays, as may be seen from the results of some experiments* on undisturbed clays by J. H. Griffith, M. Am. Soc. C. E.

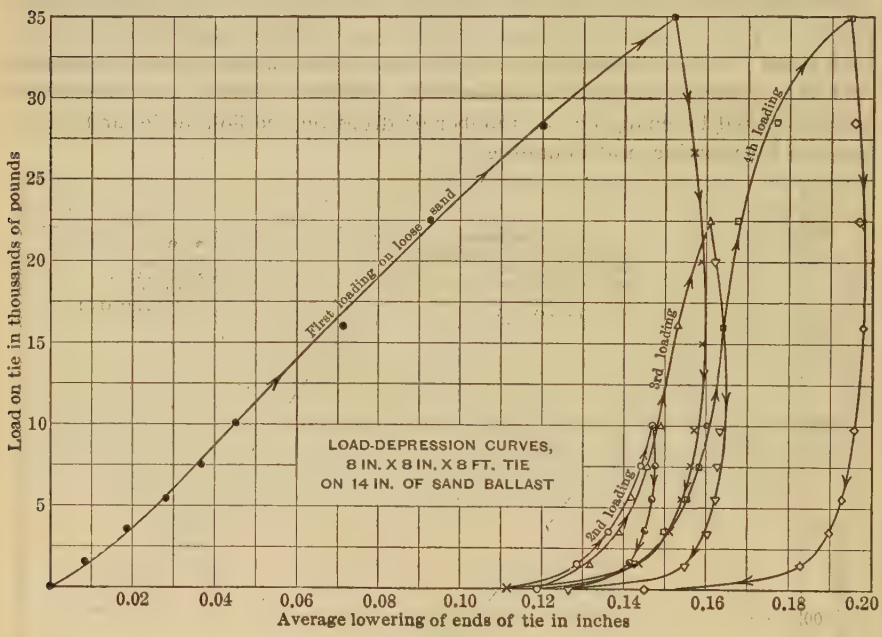

FIG. 11.

The following explanation is given to account for the tendency for the depression to increase as the load is partly removed from granular material. In the Second Progress Report of the Special Committee on Stresses in Railroad Track, an expression (Equation 18),† was derived for the intensity of pressure at any point under a tie which is just on the point of being forced

* Progress Report of the Special Committee to Codify Present Practice on Bearing Value of Soils for Foundations, etc., Figs. 10, 11, and 12, Appendix IV, *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, pp. 571 and 572.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1508.

into the ballast. Curve 1 in Fig. 12 indicates the greatest vertical components of the pressures which can exist under one-half of a tie when the intensity of pressure at the edge is p_0'. Curve 2 represents an assumed distribution of vertical components of the pressures under a tie when carrying full load. Now, assume that the load is reduced a small amount. If the ballast was perfectly elastic, the intensity of pressure at every point on the bottom of the tie would be reduced by the same amount, and the distribution of pressures would be as shown by Curve 3. The pressure at the edge has been reduced to p_0''. Curve 4 shows the greatest vertical unit pressures which can exist beneath the tie when the intensity of pressure at the edge is p_0''. Curve 3, however, lies above Curve 4 between A and B, which is an impossible condition. Hence, some of the sand below AB will flow toward the edge of the tie, causing a re-adjustment of the pressures under the tie and allowing it to settle. If the settlement due to the re-adjustment is greater than the elastic recovery, there will be an increase of depression on a removal of a part of the load.

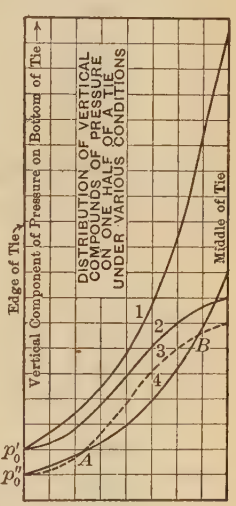

Fig. 12.

The results of some experiments in which the depression of a tie was measured during three hundred applications of load, are of interest. Three ties were used, spaced 18 in., center to center, and resting on 18 in. of sand ballast. By means of a system of levers, each of the outside ties was given half as much load as the middle tie. The middle tie was loaded to 19 750 lb. and the outside ties to 9 875 lb. The loads were applied to the ties at the rail lines, and the depressions of the tie were measured at the ends, 15 in. from the ends, and at the middle of the tie. Fig. 13, shows the total movement of the tie 15 in. from the ends, and Fig. 14 shows the elastic recovery at different points along the tie. The striking result of the experiments is that the elastic recovery is shown to be almost constant throughout the experiment. At the beginning, the ballast was very loose, as is shown in Fig. 13 by the large permanent depressions for the first loadings. At the end of the experiments, the ballast had become very compact so that when the ties were removed the sand below the tie was found to be almost as hard as rock.

The effect of jarring on the transmission of pressure through ballast is shown in Fig. 15, which gives the results of some experiments with sand ballast 24 in. deep. Load was applied to three ties spaced 20.5 in., center to center, by means of short lengths of rail. Pressures were measured 24 in. below the bottom of the tie, on the center line of the track. It is shown how load may be transferred from one tie to another by jarring or pounding. In railroad track, the ties under the rail-joints are more severely jarred than the other ties, which is one reason that the ties under the joints need the most tamping to keep them to grade.

The effect of loosening the sand under the loaded area on the transmitted pressure is shown in Fig. 16. The full line indicates the distribution of pres-

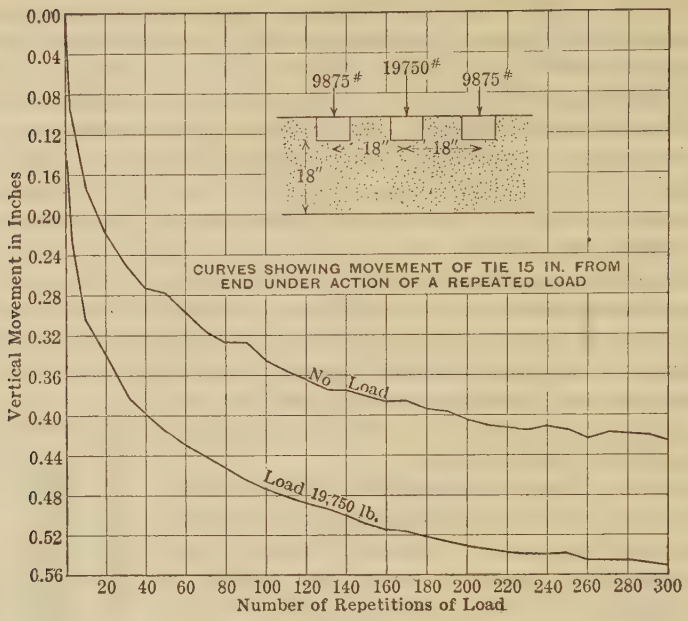
9875 #
19750 #
9875 #
18"
18"
18"
CURVES SHOWING MOVEMENT OF TIE 15 IN. FROM END UNDER ACTION OF A REPEATED LOAD
No Load
Load 19,750 lb.
Vertical Movement in Inches
0.00
0.04
0.08
0.12
0.16
0.20
0.24
0.28
0.32
0.36
0.40
0.44
0.48
0.52
0.56
20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
Number of Repetitions of Load

FIG. 13.

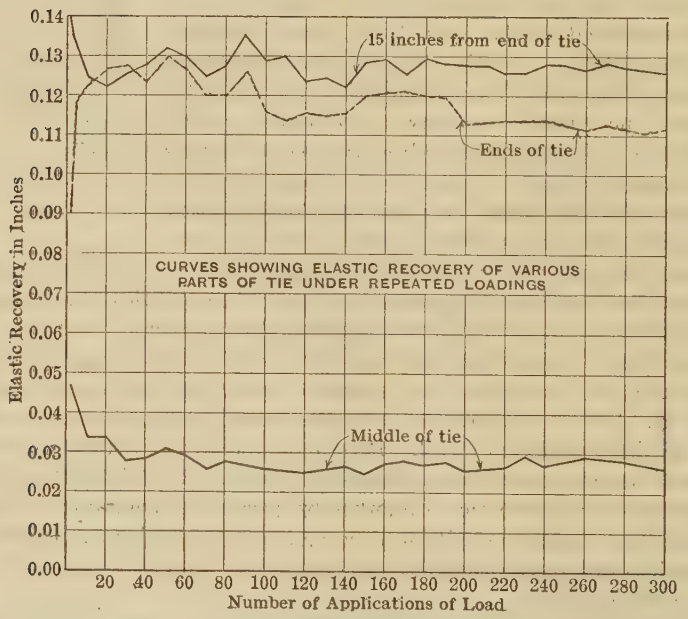
15 inches from end of tie
Ends of tie
CURVES SHOWING ELASTIC RECOVERY OF VARIOUS PARTS OF TIE UNDER REPEATED LOADINGS
Middle of tie
Elastic Recovery in Inches
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
Number of Applications of Load

FIG. 14.

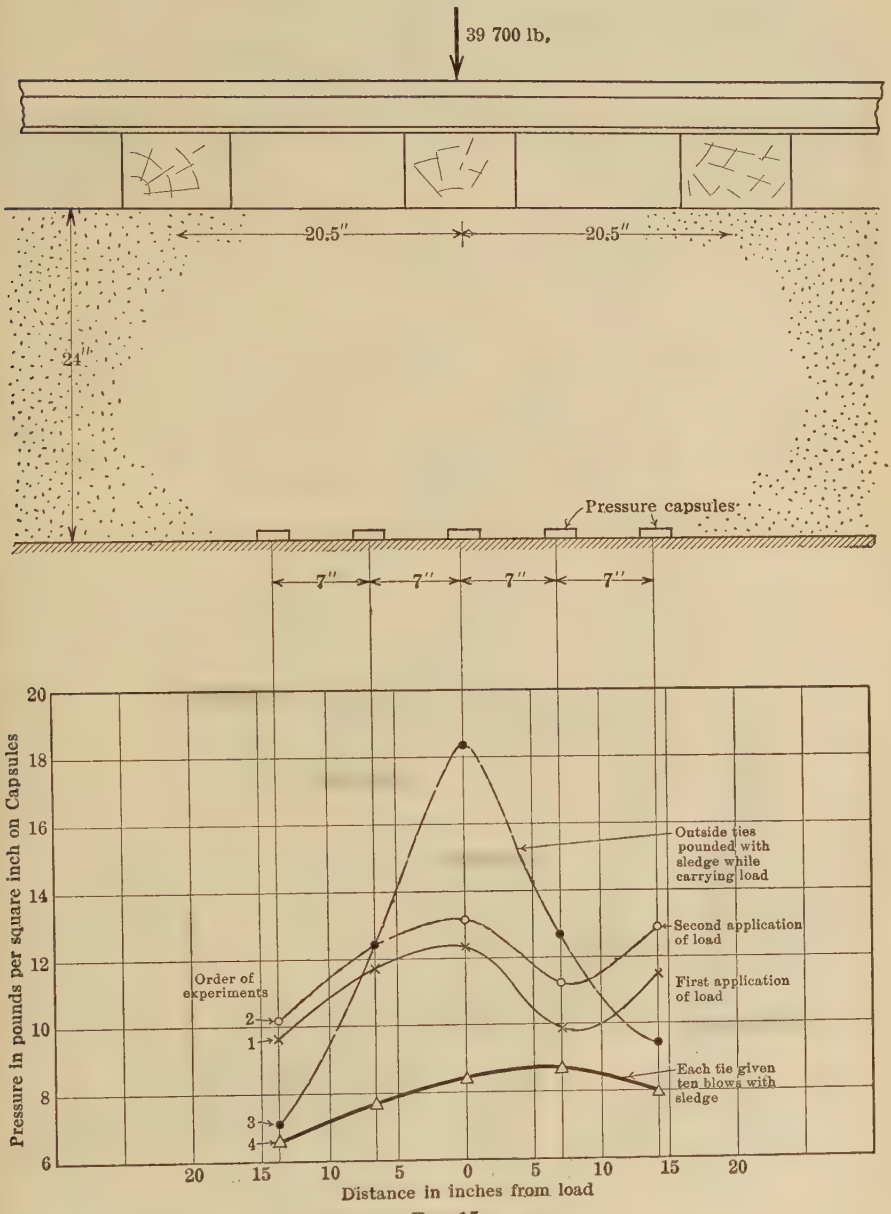
39 700 lb.
20.5"
20.5"
24"
Pressure capsules
7"
7"
7"
7"
Pressure in pounds per square inch on Capsules
20
18
16
14
12
10
8
6
Outside ties pounded with sledge while carrying load
Second application of load
First application of load
Each tie given ten blows with sledge
Order of experiments
2
1
3
4
20
15
10
5
0
5
10
15
20
Distance in inches from load

FIG. 15.

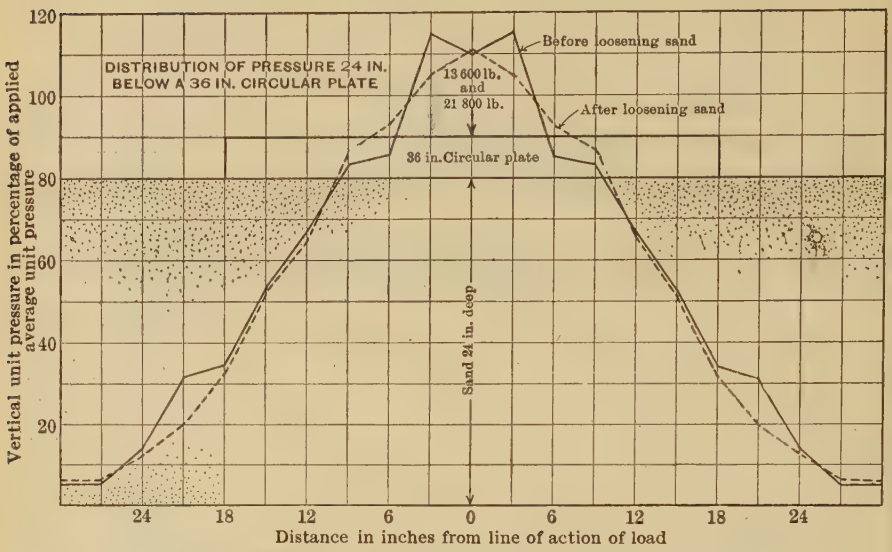
DISTRIBUTION OF PRESSURE 24 IN. BELOW A 36 IN. CIRCULAR PLATE
13 600 lb. and 21 800 lb.
Before loosening sand
After loosening sand
36 in. Circular plate
Sand 24 in. deep
Vertical unit pressure in percentage of applied average unit pressure
120
100
80
60
40
20
24 18 12 6 0 6 12 18 24
Distance in inches from line of action of load

FIG. 16.

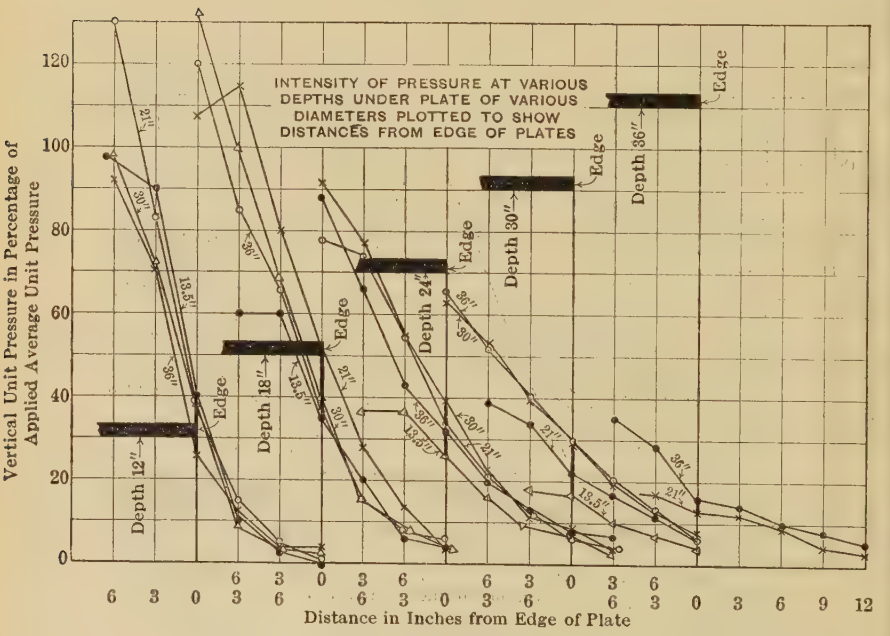
INTENSITY OF PRESSURE AT VARIOUS DEPTHS UNDER PLATE OF VARIOUS DIAMETERS PLOTTED TO SHOW DISTANCES FROM EDGE OF PLATES
Vertical Unit Pressure in Percentage of Applied Average Unit Pressure
120
100
80
60
40
20
0
Depth 12"
Depth 18"
Depth 24"
Depth 30"
Depth 36"
Edge
6 3 0 3 6 6 3 0 3 6 6 3 0 3 6 6 3 0 3 6 9 12
Distance in Inches from Edge of Plate

FIG. 17.

sure 24 in. below a 36-in. plate. The plate was then removed, the sand was loosened to a depth of 10 or 12 in., and the plate was replaced. The dotted line shows the distribution of pressure after the sand was loosened. The difference in the results is due to the difference in compactness of the sand below the center of the plate. The experiments on the distribution of pressure under a 36-in. plate through 24 in. of sand had been preceded by a series of experiments with sand 30 in. deep, using plates 36 in., 30 in., 21 in., and 13.5 in. in diameter, and no attempt had been made to loosen the sand between experiments.

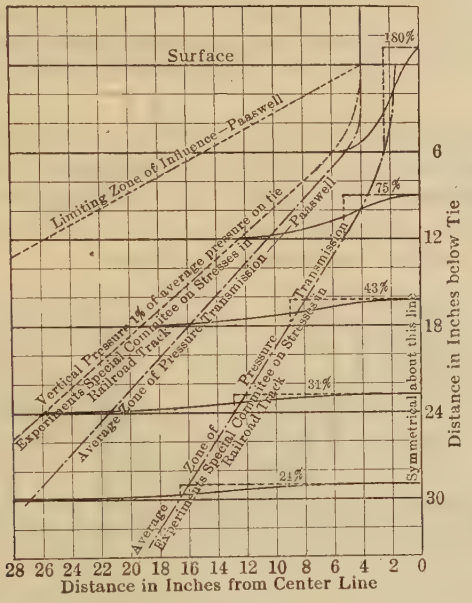

FIG. 18.

The results of some experiments on circular plates of various diameters is shown in Fig. 17. The transmitted pressure at different depths is plotted with respect to the edge of the plates. The results have inconsistencies of the kind indicated in Fig. 16, but serve to bring out the fact that the pressure at the various points below the edge of the plate is not much different for plates carrying the same unit pressure. Hence, the pressure directly below the edge is a governing factor in the capacity of a plate for carrying load and also in the distribution of pressure at different depths.

Fig. 18 shows the results of the experiments of the Special Committee on Stresses in Railroad Track and, in order that comparisons may be made, the results of the author's calculations, as given in Fig. 5, are also shown. It will be noted that the qualitative results are similar, but that there is a considerable difference quantitatively.

F. N. MENEFEE,* ASSOC. M. AM. SOC. C. E. (by letter).—In discussing this paper, the writer invites attention to experimental evidence which, for the most

* Prof., Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

part, substantiates the mathematical deduction therein. In 1913, after working on railroad structures for which the office standards required that a live load on an earthen embankment be assumed to spread on a slope of 1:1 or 45°, from which the intensity of vertical stress was computed at any distance below the roadbed, the writer built an experimental earthen embankment for the purpose of trying to establish the accuracy of the assumption that the load on an embankment spread on a slope of 1:1.

As far as is known to the writer, the question of a horizontal force within the earthen fill had not occurred to engineers accustomed to designing structures under fills. Because some spread to the load is assumed in all railway office standards, it now seems that engineers should have thought of this possibility which will soon be an established fact among those familiar with soil pressure.

It is fortunate that most experimental work in earth pressures, although of necessity a special case, has a rather wide and general application. Hence, the writer's experiment and the deductions therefrom may be of interest in connection with this paper, particularly, inasmuch as the approach, both in experiment and technical explanation, is different.

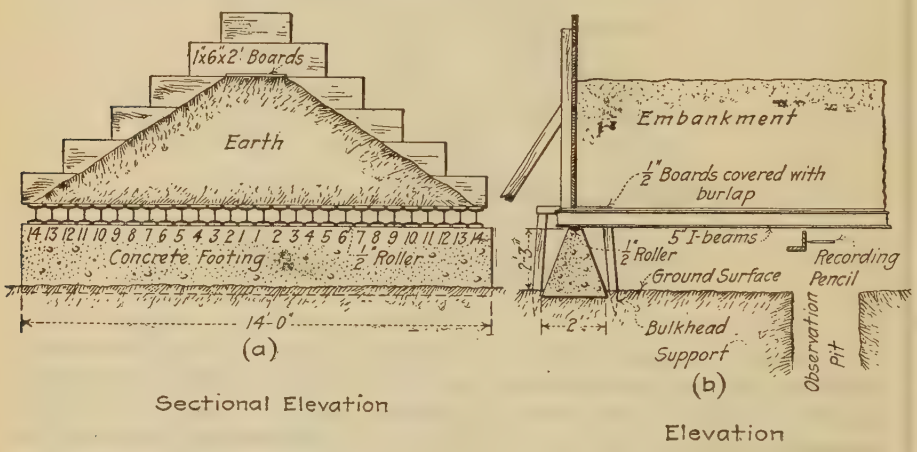

FIG. 19.

The writer built a small earthen embankment 14 ft. wide on the base, 13 ft. long, 2 ft. wide at the top, and 4 ft. high. The embankment was placed on 5-in. I-beams, the ends of which were supported by concrete piers. To prevent undue friction between the I-beam and the concrete pier ½-in. round rollers were put under each I-beam at one end.

The idea of this scheme was to provide a base sufficiently strong to sustain the fill and any superimposed loads and yet yielding enough to permit any variation in pressure at the base of the fill to manifest itself in the behavior of the beams. There were 28 beams, 14 on each side of the center line of the fill. Each I-beam had a pencil attached to it, such that a downward or a lateral

motion of the beam was recorded on a piece of stiff paper. (See Fig. 19 (*a*) and (*b*)).

As a result of loading the embankment both by impact (falling weights from various heights) and by static loads, it was established that the two beams on each side of the center line of the roadway (6 in. apart) moved either straight downward or downward with only a slight slope outward, and that the horizontal component of the movement of all beams increased as the distance from the center line increased. No definite law governing the variation from vertical to horizontal could be established, but it was plain and evident on all records. (See Fig. 20.) Although the photograph (Fig. 20), is useful only in assisting one to visualize the action, Tables 1 and 2 give the amount of vertical and horizontal movement on each side of the center line of the embankment.

The ratio of the moments of inertia of the I-beams about the horizontal and vertical axes was 12.1 : 1.23. Hence, the horizontal deflection was greater than for beams as stiff in the horizontal plane as in the vertical plane. Had beams equally stiff about both horizontal and vertical axes been used, the horizontal deflections would be considerably less than they appear. To obviate this

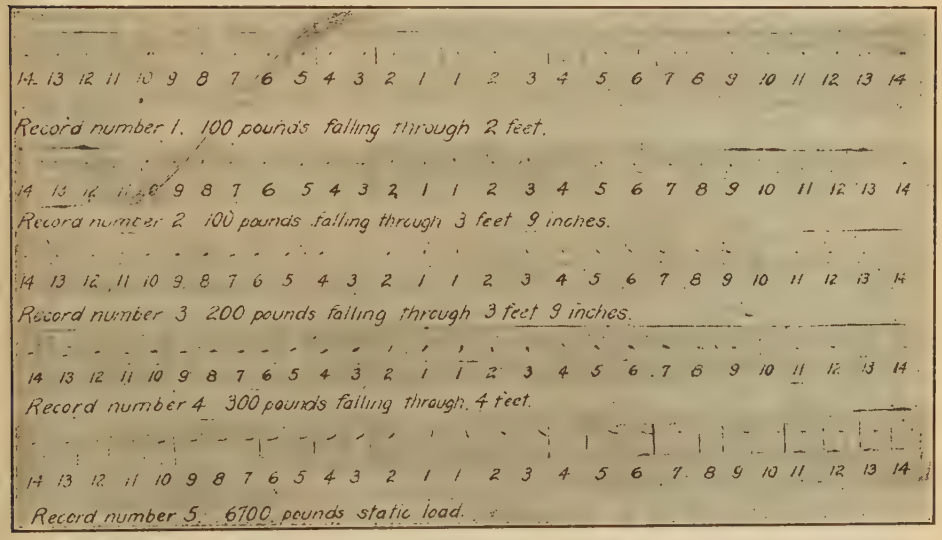

FIG. 20.—PENCIL MARKS MADE BY PENCILS DURING TESTS.

for two typical records, Tables 1 and 2 are inserted. In these tables, the vertical and horizontal deflections of Records Nos. 4 and 5 were carefully magnified, measured, and tabulated. As deflection varies inversely as the moment of inertia, the beams have a magnified horizontal deflection of about 9.83 times. By dividing the values for horizontal deflection by 9.83, a figure was obtained which represents the horizontal deflection of a beam, the stiffness of which in the horizontal plane equals that in the vertical plane. Table 1 taken from Record No. 4 (an impact record) shows that as far out as the ends of the

TABLE 1.—COMPARATIVE HORIZONTAL AND VERTICAL DEFLECTIONS BASED ON RECORD NO. 4.

Beams to Left of Center Line of Embankment.														
14	13	12	11	10	9	8	7	6	5	4	3	2	1	Beam No.
0.01	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02	0.03	0.033	0.045	0.075	0.076	Vertical deflection, in inches.
0.08	0.09	0.08	0.06	0.08	0.08	0.10	0.09	0.09	0.10	0.10	0.10	0.05	0.07	Horizontal deflection, in inches.
0.0081	0.0091	0.0081	0.0060	0.0081	0.0081	0.0101	0.0091	0.0091	0.0101	0.0101	0.0101	0.0057	0.0071	Horizontal deflection, divided by 9.83.

Beams to Right of Center Line of Embankment.														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	Beam No.
0.08	0.06	0.08	0.07	0.06	0.05	0.035	0.027	0.02	0.025	0.01	0.01	0.01	0.00	Vertical deflection, in inches.
0.05	0.01	0.03	0.06	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.08	0.08	0.06	Horizontal deflection, in inches.
0.0057	0.0010	0.0030	0.0060	0.0081	0.0081	0.0081	0.0081	0.0081	0.0091	0.0091	0.0081	0.0081	0.0060	Horizontal deflection, divided by 9.83.

records, the vertical deflection would be at least equal to, if not larger than, the horizontal deflection, and Table 2 from Record No. 5 (a static load), just as definitely shows a possibility of vertical deflection out to the toe of the slope. It must be remembered that these deflections, both horizontal and vertical, are directly the result of the load on the top of the embankment corresponding to the live load, and that the effect is appreciable out to the toe of the slope.

Regardless of the fact that the I-beams gave less resistance to horizontal movement than to vertical movement, it is evident that the vertical load on the fill became a sloping load before it reached the base of the fill. The cross-section of the roadway showing the direction of the stresses is given in Fig. 21 where the lines fanning downward and outward indicate the principal stresses that cause the beams to deflect, and the line, *a-b*, drawn at right angles to them, is the other principal stress. Mr. Paaswell states that these stresses are of opposite sign, that is, those running from the top to the bottom of the embankment, or any chosen horizontal plane below the top, are compressive, whereas those at right angles are tensile. Although the writer agrees with the first statement, the latter is not entirely evident from the following analysis:

FIG. 21.

TABLE 2.—COMPARATIVE HORIZONTAL AND VERTICAL DEFLECTION BASED ON RECORD NO. 5.

Beams to Left of Center Line of Embankment.														
14	13	12	11	10	9	8	7	6	5	4	3	2	1	Beam No.
0.00	0.00	0.00	0.00	0.012	0.015	0.018	0.022	0.029	0.037	0.052	0.068	0.041	0.091	Vertical deflection, in inches.
0.05	0.06	0.055	0.05	0.08	0.08	0.118	0.12	0.12	0.095	0.110	0.10	0.085	0.02	Horizontal deflection, in inches.
0.0050	0.0061	0.0055	0.0050	0.0081	0.0081	0.0120	0.0122	0.0127	0.0096	0.0112	0.0101	0.0086	0.0020	Horizontal deflection, divided by 9.83.

Beams to Right of Center Line of Embankment.														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	Beam No.
0.089	0.074	0.063	0.055	0.015	0.027	0.02	0.014	0.02	0.017	0.022	0.013	0.01	0.008	Vertical deflection, in inches.
0.03	0.065	0.10	0.12	0.117	0.115	0.06	0.09	0.08	0.07	0.07	0.065	0.057	0.03	Horizontal deflection, in inches.
0.0030	0.0066	0.0101	0.0122	0.0119	0.0117	0.0061	0.0091	0.0081	0.0071	0.0071	0.0066	0.0058	0.0030	Horizontal deflection divided by 9.83.

Considering any particle, A, in the embankment to either side of the flat portion of the roadbed (Fig. 22), with top and bottom parallel to the natural slope of the earth and sides vertical, Rankine's theory states, and inspection shows, that there will be conjugate stresses on it as indicated by the arrows. Removing the particle as in Fig. 23 (a), subdividing, and putting into equilibrium, Fig. 23 (b), decomposing the forces on the bottom and side into vertical and horizontal components, Fig. 23 (c), converting the particle into a cube, Fig. 23 (d), and finally subdividing again by another diagonal and putting in equilibrium, Fig. 23 (e), shows that a principal stress acts perpendicular to the diagonal $d\ l$. The amount and direction of this principal stress will depend on the relation of the stresses in Fig. 23 (d). Rankine's theory applied here makes the stress, v, Fig. 23 (d), greater than the stress, h, and, considering cohesion, the ratio is even greater. The shearing stresses are of equal intensity, hence h and v combined with the shear stresses on the particle cause P to have a slope less than 45° with the vertical and a magnitude greater than either h or v, as shown by the force polygon, Fig. 23 (f).

Again, considering a number of small cubes side by side across a section of the embankment as at BC in Fig. 22, all cubes to the left of FG may be considered as subject to stresses like those in Fig. 23 (d). Between FG and HJ, however, the situation is changed. It seems reasonable to suppose that the

particle immediately in the middle is subject to a vertical pressure on top and bottom, and that it is subject to equal horizontal pressures by the adjacent particles on either side, or an infinitely small particle in the center of the embankment is in equilibrium but not subject to shear. However, as we consider particles nearer the lines, *FG* or *HJ*, it seems reasonable to state that the lateral supporting pressure from adjoining outside particles decreases. Hence, everything above the line, *BC*, would tend to slide away from the center. As long as equilibrium obtains, however, there must be a resistance to the outward sliding, coming from the planes on which that mass rests. This resistance is a frictional or shear stress directed toward the center of the embank-

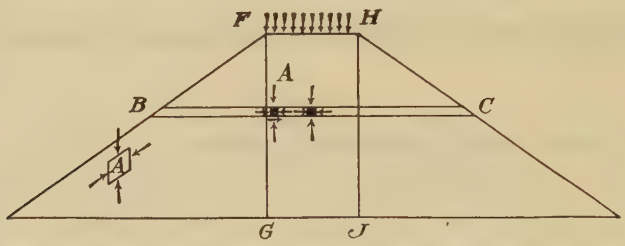

FIG. 22.

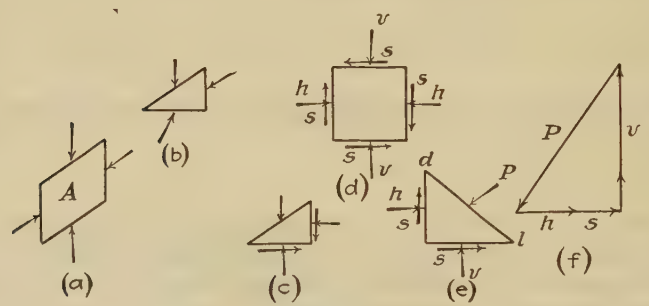

FIG. 23.

ment along the bottom of all particles to either side of the center, which increases such that its total equals the amount of decrease in the lateral supporting pressure, and if to the left of the center is in the same direction as that along the bottom of Fig. 23 (*d*), and as shown again in Fig. 22 at *A*. Thus, by another line of qualitative reasoning, there are lines of principal stress, *P*, in the direction of that in Fig. 23 (*e*) anywhere to the left of the center line of the embankment, and instead of arching as in Fig. 24, the lines of stress take the shape of those in Fig. 21, with the greatest vertical stress immediately in the center of the cross-section, and decreasing as the toe of the slope is approached. Inasmuch as there are always two principal stresses 90° apart in direction, it follows that as there are principal stresses varying in direction from the vertical to the slope of the embankment, there are corresponding principal stresses varying from the horizontal at the center line of

FIG. 24.

the section to a direction perpendicular to the slope of the embankment at the surface of the slope, Fig. 21.

Although the foregoing discussion makes use of shear in showing the cause and behavior of the principal stresses, it is believed that no analytical expression has yet been developed, which includes both direct stress and shear.

As mentioned previously, it does not follow from this analysis that tension need be anywhere in the embankment, except possibly at the vertical center line through the load. It is probable that with insufficient lateral earth or material to resist the horizontal component of the principal stresses by shear on the lower strata, a center crack might develop as a result of shrinking in the material after it had expanded laterally, due to a superimposed load. In general, however, it is the writer's opinion that as long as the load and the quasi-elastic properties of the soil exist, there is no tension in earth loaded as in this problem.

In order to show the variation in vertical pressure, the vertical component of Record No. 5 was carefully measured and plotted (Fig. 25). It is believed

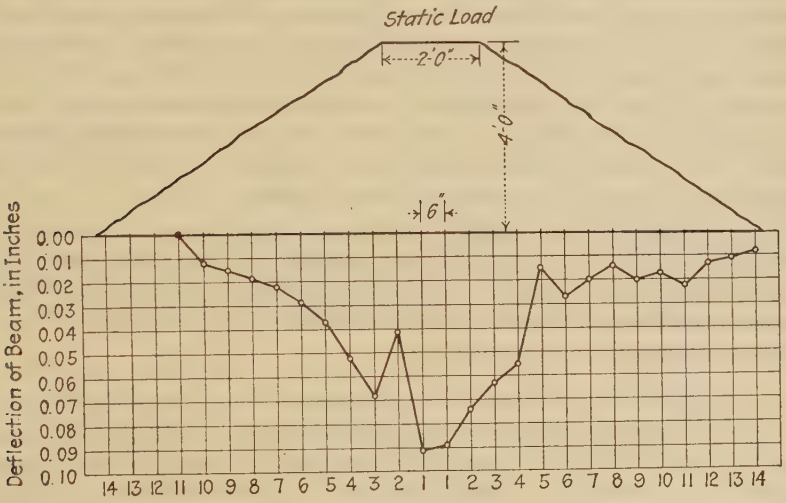

FIG. 25.

that Record No. 5 is typical. The area enclosed by the bounding curve resembles a triangle rather than a parabola, as has been suggested. It is not exactly what is suggested in Second Progress Report* of the Special Committee to Report on Stresses in Railroad Track, but it must be borne in mind that its analysis is based on direct transmission of stresses by means of compression and that shear is neglected.

In the writer's opinion, the essential points established with reference to soil pressures in embankments are as follows:

1.—In embankments of any size, the greatest intensity of vertical pressure along the base of the section is under the center of the load or at the line of symmetry of the section for a single-track roadbed.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1409.

2.—The vertical intensity of pressure due to live load decreases on either side of the point of greatest intensity, but extends all the way to the toe of natural slope. As shown by these tests, however, the vertical intensity was comparatively small beyond the eighth I-beam on either side of the line of symmetry. If the same ratio held for higher embankments the live load could be said to be spread over areas as wide as twice the height of the embankment plus the width of the roadbed, with the intensity varying from a maximum at the center to almost zero at the outer boundaries of this area. That this variation is uniform is hardly probable, although it certainly is more correct than to assume that the pressure is uniform, even though spread over a greater area than at the top of the embankment.

3.—On all horizontal planes there is a stress directed away from the center or from that point where the greatest principal stress is vertical. Tables 1 and 2 show this horizontal stress to be greatest at about Beam No. 8, or at a point near where a 45° line from the end of the tie strikes the base of the fill. Table 2 shows that the maximum vertical deflections are 0.36 in. for Beam No. 1 to the left and 0.40 in. for Beam No. 2 to the right. These values are 8.5 and 10.4 times greater than the maximum horizontal deflections taken from the reduced horizontal deflection column. Likewise, in Table 1, the maximum vertical deflections are 0.07 in. for Beam No. 1 to the left, and 0.08 in. for Beam No. 1 to the right, these values being 6.9 and 9.8 times greater than the maximum horizontal deflections taken from the reduced horizontal deflection column. From this, the writer believes the maximum horizontal stress may be taken as one-tenth of the maximum vertical stress.

In conclusion, it might safely be said that any load on any kind of earth is spread in much the same way as in the embankments, and that earth particles under the load are compressed vertically, expand horizontally, and through cohesion and shear and normal pressure transmit part of their load to adjacent particles in the same horizontal plane.

E. G. Haines,* M. Am. Soc. C. E. (by letter).—This paper can hardly fail to be of interest to that large body of engineers doing work requiring excavations to a considerable depth, designing foundation structures, or structures for the support of earthwork. As far as the writer is aware, this is the first attempt to bring together and show the close relationship between the various formulas relating to elastic solids, and apply them to soils.

The necessity of considering soils as an elastic body rather than an hydrostatic or cohesionless mass, was pointed out by the writer in 1907, in a discussion† of a paper by J. C. Meem, M. Am. Soc. C. E. In that discussion, the writer believes he first laid down the underlying simple, fundamental law governing the fracture of unsupported earth in excavation, which law is nothing more nor less than the development, on any surface, of the greatest possible volume having the smallest possible surface of resistance. Although this principle has been before the Profession for fifteen years, it is as yet uncontroverted.

* Asst. Div. Engr., Transit Comm., New York City.

† "The Bracing of Trenches and Tunnels", *Transactions*, Am. Soc. C. E., Vol. LX (1908), pp. 27 and 29.

Assuming the soil to be homogeneous in all respects, the principle at once gives, as demonstrated at the time, a definite form to the fracture, and the prediction was made in the discussion that a cut in solid rock, if carried to sufficient depth, would fracture in the same manner as the less compact soil. H. G. Moulton,* M. Am. Soc. C. E., has shown that the statement is true in rock excavation at depths of 500 or 600 ft.

Although the writer's theory was based on homogeneous material, it is recognized that soil is seldom approximately such, and it is of interest to note the close agreement as to form of the fracture of unsupported earth in a bank, as shown by the writer's Fig. 29† with the author's Fig. 7, showing the paths of motion of the grains of soil, using variable factors. As pointed out by the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations,‡ the two elements in soil having, probably, the greatest influence on its elastic and plastic qualities—and, therefore, its resistance under loads—are the amount of colloidal matter and quantity of water present. Both these elements vary through quite a wide range and, until experimental research has determined the amount and range of such influence, no quantitative results can be obtained from the use of such influence lines as the author has presented; it is to be hoped, therefore, that exhaustive experiments may be undertaken along both the lines suggested.

In the meantime, the Society is indebted to the author for presenting the results of his studies along qualitative lines, and it is to be hoped that the subject will be further analyzed and discussed as there is little doubt that the lines of stress in soil quite closely follow those in more perfect solids. It is only by such study and research that the work of designing foundations, dams, and many other structures, can be placed on a plane comparable with their importance, instead of the haphazard guesswork now only too common.

George Paaswell,§ M. Am. Soc. C. E. (by letter).—In limiting the text of the paper to the transmission of pressure through solids and soils, the writer had purposely excluded discussion of earth-pressure theories. It is rash to attempt a mathematical analysis of a topic of so controversial a nature. A soil mechanics is now in process of slow development. One important point has been noted, negative in character, namely, that soils vary so much in fundamental characteristics, that a universal mathematical treatment, quantitative in nature, is almost impossible. For the time being, progress lies through individual treatment. A method, predicated on grain analysis, lies, most likely, at the other extreme of quantitative research. An elastic analogy proves fruitful in developing some simple theorems of pressure transmission, but one would be rash indeed, who would build a rigorous soil mechanics on so precarious a foundation.

A few words may not be amiss about earth-pressure theories in general. The plane of rupture has been a pleasing fiction on which to found present theorems

* "Earth and Rock Pressures", *Transactions*, Am. Inst. of Mining and Metallurgical Engrs., Vol. 63 (1920).

† *Transactions*, Am. Soc. C. E., Vol. LX (1908), p. 30.

‡ *Proceedings*, Am. Soc. C. E., February, 1921, Papers and Discussions, p. 9.

§ Section Engr., Public Service Comm., First Dist., New York City.

of earth pressure. It is a simple assumption, and, as is generally recognized, has no basis in actual practice. A soil that acquires, or already possesses, elastic properties cannot, at the same time, yield planes of rupture. It is admitted, as the result of much painstaking analysis and experimentation, that soils do possess a certain degree of elasticity. A soil that possesses no elastic properties, as a whole (although its individual grains may be, to a high degree, elastic structures) should and generally does give a plane of rupture. As this is the limiting and maximum case of lateral pressure, the engineer who predicates his retaining-wall pressure on a standard plane-of-rupture theory, is providing a safe margin of load.

Dr. Charles Terzaghi has noted (in some papers which it has been the writer's privilege to read) that the elastic properties of a solid may be replaced, hypothetically, by a surface hydrostatic equivalent loading on discrete granular particles, so that the granular material of Coulomb and Rankine may be taken as the limiting case of an elastic solid. Such an analogy serves well to correlate the elastic and granular materials of engineering practice. To conclude, engineers may well cease to seek the true mathematical expression for lateral earth pressures on sustaining structures. As the writer has pointed out, the methods of analysis of Coulomb and Rankine serve as asymptotic expressions to exhibit extreme cases of pressure. The engineer does well to use them when more exact analysis is likely to prove fruitless.

Taking up some of the points noted by Mr. Meem, the writer would state that experimenters have demonstrated the elastic behaviour of soils. That such elastic phenomena are not comparable with those exhibited by the usual structural materials, such as wood, concrete, or steel, goes without saying. A modulus of elasticity of soils is undoubtedly a function of the pressure and is vitally dependent on the viscosity and other molecular properties of the soil. The presence of water naturally affects, most profoundly, its stress-strain behaviour, but, qualitatively speaking, the elastic analogy still holds.

As to Mr. Meem's "inverse" hydrostatic law, it is recognized that the pressure on sheeting in an ordinary cut, not excessively deep, is a matter of local disturbance. A well-timbered cut, properly keyed, may exhibit no pressure except that produced by the wedges. A local disturbance may produce a pressure at that locality, and as the upper levels are more prone to such disturbance than the lower ones, symptoms of lateral pressure may be expected to be found first at the top of the cut. Mr. Meem errs, in the writer's opinion, in founding a law for the distribution of a pressure, so general in nature, on such obviously local phenomena. Arching, etc., may be expressed in the terminology of the paper as phenomena due to surfaces of compression, etc. It is interesting to analyze the distribution of pressure around a tunnel periphery and to note how well the elastic analogy theorems yield results of quantitative worth. The writer will not burden the present discussion with such analysis.

Professors Enger and Menefee have noted that the principal stresses in granular material should both be compressive. When the writer made the attempt to adapt the theorem of Michell to the work at hand, it seemed that the horizontal principal stress should be a tensile one, as this was most probably the character of the stress near the central core of the loaded area. On further

consideration, it has been decided that the elastic analogy of Michell's theorem may be carried to its full, utilizing the result that the principal stresses are alike in character, and are both compressive. It is gratifying to note that Professors Enger and Menefee concur in this.

Briefly stated, the transmission of pressure is downward and away from the load. The modifications of the theoretical loadings as found in actual practice may well be as stated by Professor Enger. Qualitatively, the results presented in the paper are not materially affected. That the percentage of voids in a granular material is a fundamental factor in its pressure transmission properties may be noted at once when it is realized that such pressure transmission, with its accompanying distortion of soil, is by means of intergranular movement and adjustment and not by distortion of the grains. Dr. Terzaghi has pointed out that great pressures on confined sands hardly change the proportion of voids, whereas light shaking will materially decrease the voids. The percentage of voids controls the freedom of movement and, consequently, the strength.

Obviously, one would not apply Hooke's law, quantitatively to a granular material. The load deformation curves do show a characteristic relation, however, with proper allowance for the "hysteresis" effect, and the elastic methods may be followed with due appreciation of their limitations.

Professor Menefee's experiments on a railroad embankment seem to indicate that the writer's stress distribution curves correctly exhibit the manner of pressure distribution at various horizontal layers and emphasize his statement that a 1 : 1 slope does not prove a safe rule for the limit of pressure disturbance. There is a fallacy in attempting to give a numerical relation between maximum horizontal and vertical pressures. That is a heritage from the Rankine theorems. There is no reason why a numerical relation of constant value, should exist between the horizontal and vertical pressure, except in the isotropic materials. Even in this case, the Poisson ratio is of questionable worth. Reliable data are to be found for this ratio for only one substance, namely, water, for which the ratio is unity. The writer would emphasize the fallacy of this quest for a ratio, as it is on this search for a lateral, vertical pressure ratio, that most theories of lateral earth pressure are foundered.

In preparing the first draft of the paper, the writer pointed out that the elastic analogy was not a new discovery and that Mr. Haines had indicated it several years ago. In preparing a new draft, the writer, through oversight, omitted mention of this fact, and is pleased to call attention to such pioneer work. Mr. Haines has stated that the circle fulfills the problem of minimum surface to maximum contents as far as surface of slip is affected, but the writer hesitates to admit such a type of surface. There is an analogy which occurs to him, in the determination of the curve of swiftest descent between two points not in the same vertical, a famous problem in the calculus of variations. Here, it is shown that the curve is one of the family of cycloids, and this type may be that sought by Mr. Haines. This is a point of minor importance and the agreement between the curves developed by mathematical analysis and by the empiric method of Mr. Haines is striking.

In conclusion, the writer would point out that the methods suggested and the elastic analogy developed are merely a step in the development of a rational soil mechanics which is not to be confounded with a general law for the distribution of soil pressure. Its function will be the correlation of all soil phenomena in accordance with definite soil factors, possibly as has been indicated, with void ratio, moisture content, sieve ratio, and, perhaps, with certain molecular properties such as friction, viscosity (if there is a distinction between friction and viscosity phenomena), and the like. Patient experimentation will produce such correlation, and the near future seems to be very promising in this regard.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

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Paper No. 1507

ADDRESS AT THE ANNUAL CONVENTION, HOTEL WENTWORTH, NEAR PORTSMOUTH, N. H., JUNE 21ST, 1922.

By John R. Freeman,* President, Am. Soc. C. E.

The By-laws of the Society require an address by the President at the Annual Convention. In nearly all scientific societies, the address of the President comes at the close of his term, which seems a more appropriate time. I awoke to this time of delivery too late to permit the development of my chief topic as I have desired. Before deciding on this topic, and in order to learn the traditional scope and treatment, I read all the Presidential Addresses of the past 25 or 30 years. I found them mostly devoted to one of three subjects: (1) The Society; (2) the status and relations of the Engineer to the Public; and (3) a historical review of progress in that line of engineering to which the speaker had chiefly devoted his professional life.

While thus studying the proper topic and scope for a Presidential Address, my thoughts have been drawn in two directions: One, Progress in the Development of this Society and Its Possibilities for Greater Usefulness, and the other the Development of Hydraulic Science. I am particularly drawn to the second topic, because it is now almost fifty years to a day since I began as a Junior Assistant Engineer in the office of the Water Power Company, at Lawrence, Mass., and my professional life has been chiefly devoted to Hydraulic Engineering.

Before proceeding with this second topic, I cannot, following forty years of membership in the Society, many years of membership in various other engineering and scientific societies, and five months in the closer view given by my present office, refrain from saying something about The Society and Its Progress.

As I recall my three years of service on the Board of Direction about 25 years ago, and compare the technical activities then and now, I find that the time and effort now given by the individual members of the Board toward the development of the Society has been vastly increased. Then, the Board could

* Cons. Hydraulic Engr., Providence, R. I.

finish its order of business in an evening, or, at most, in an afternoon plus an evening, and the proportion of Directors attending from a distance was much smaller than at present. Now, the Board requires at least two full days and an evening, four times a year, and this large amount of time is all spent in good hard work, in active discussion and thoughtful consideration of ways and means of promoting the efficiency and service of the Society to its members, and in carefully scrutinizing the records of candidates for membership, in order that the Society may do its share in maintaining the status of Engineering as one of the learned professions.

A few months ago, I was disposed to think it somewhat ridiculous that a Board meeting of the Society should take so many more hours than the Board meeting of a great business corporation, but now I see from the inside that this is not unreasonable. Settling matters in the Board meetings, instead of debating them in a general meeting, saves a lot of time to the members. The members of the Board represent different sections of the country having different problems and all of them are men of strong convictions, trained by their profession and by their daily tasks into a sense of responsibility. Many difficult questions of policy arise. I have heard suggestions from outside, more in jest than in seriousness, that so much time spent in meetings indicates that the Board is a divided body which spends too much time in "scrapping" and on petty detail, and so I will say that, in nearly a half year as Chairman, there has been not one word of acrimonious discussion, not the slightest manifestation of sectionalism, nothing but earnest forward-looking consideration of how best to achieve the purposes set forth in the Constitution. The 10 500 members should appreciate the spirit in which the busy men who constitute the Board of Direction leave their important responsibilities and give their valuable time to these matters. Twenty-six out of the twenty-eight Directors were present at the meetings at Dayton, Ohio, on April 3d and 4th, 1922, and twenty-four Directors are present at this Annual Convention, three of whom came from the Pacific Coast. The Board thus functions from a broad National viewpoint, while the majority of members present at the mid-winter meeting, or at the midsummer convention, always are from relatively near-by.

All members may well be pleased with the Society's progress as shown in the character and extent of the papers published during the past year, in the steady growth in membership, in the activities of 34 Local Sections scattered through almost every important section of the country, in the 45 Student Chapters—one at substantially every important engineering school—and in the arrangements for Technical Divisions specializing in Power Development, Sanitary Science and Public Health, Irrigation Engineering, and Highway Engineering, provided for at the meeting of the Board of Direction held on June 19th, 1922. It is intended that, henceforth, when "birds of a feather flock together", there will be a place for their flock inside the Society, with no need for them to form a little new society outside. One of the many pleasing features of the Dayton meeting was the innovation of the presence, by invitation, of a large group of students from two of the Ohio Engineering Colleges, whose professors had suspended their lectures, in order that the students might attend the technical

session held at that meeting. We hope this may be repeated in each State where we meet, wherever there is sufficient seating capacity in the convention hall.

Above all, the members should be pleased with the earnest work for the advancement of engineering arts and science now being conducted by the Special Committees. I fear that too few appreciate the time now being given to this committee work by busy men—the most eminent and best fitted men that we can find in the United States, or in Canada, for each special task—at great personal sacrifice of their time and comfort, and in earnest spirit of service to the Engineering Profession. The payment of the traveling expenses of the members of these Committees to a common meeting place, although it makes something of a hole in our budget, is a mere pittance in proportion to the service rendered to the public, to engineering, and to the members of the Society collectively. In the rapid development of Engineering, this kind of committee work may well be increased, and the Committee on Special Researches, of which A. N. Talbot, Past-President, Am. Soc. C. E., is Chairman, is working to get together half a dozen new committees in special fields.

The Constitution states that the main objects of the Society are:

(1).—"The Advancement of Engineering Science". This is done mainly by committee work such, for example, as that on structural standards, and by contributing data and experiences in papers.

(2).—"The Professional Improvement of Members". This is done through the maintenance of high standards for the admission of members and by bringing the Junior into friendly contact with the Senior.

(3).—"The Encouragement of Intercourse". As seldom more than 3 to 5% of the 10 500 members, scattered all over the United States and throughout the world, attend either the Annual Convention or the Annual Meeting, and as less than 1% attend the monthly meetings, this intercourse obviously must come mainly through Local Sections and through the published *Proceedings,* or perhaps through a new bi-monthly, containing more matters of current interest to members and to the younger members in particular.

(4).—The fourth great objective stated in the Constitution, "The Establishment of a Central Point of Reference for the Preservation of Data", is obviously met by the share of this Society in the establishment and maintenance of the great Engineering Societies Library, but because of the wide distribution of membership, this fourth objective must be mainly attained through the files of *Transactions* in each member's office or home.

Let us now briefly consider possibilities of greater service by the Society toward the better attainment of each of these four objectives.

In Engineering, Science often lags far behind the Constructive Arts. Watt built engines without knowing much about thermodynamics. Arches and trusses long preceded stress diagrams. Portland cement has been used one hundred years, and its chemistry is not yet well understood, and great bridges have been built with little knowledge of the metallurgy of steel. The pioneer is, of necessity, often wasteful with his factor of safety, or knowing the risk must, of necessity, take a chance, as with the early sheet-iron water pipes of the

California miners, and "through our failures we achieve success". Then Science comes and shows the way to economy, reducing wasteful cross-sections, combining experiences, deducing theory from successful practice, and giving confidence for projecting structures to longer spans, higher pressures, and new lines of attack. Textbooks and published data often lag far behind the information collected here and there by some one under stress of circumstances. The *Proceedings* and *Transactions* of Engineering Societies provide convenient places in which to record these forward steps.

One great service of the *Proceedings* of technical societies is the collection, preservation, and early dissemination of these experiences and newly discovered facts, so that they may be available to engineers and to writers of textbooks.

The Society machinery does not yet seem particularly well organized for scanning the horizon, discovering, and systematically bringing foward and publishing these new data. The busy worker has his hours so full of responsibilities that commonly pressure must be brought on him to sacrifice his time to prepare a paper, through the appeal of rendering service to others. Some of the best papers of the past year have been thus obtained, more can be done in this line and Object No. 1 of the Constitution thus better attained.

The Engineering Foundation was established to aid in advancing Applied Science and in developing new data, by a far-seeing engineer who intended his own gift to be merely a nucleus to which others by adding might testify in a substantial way to the help they had received from association in the Engineering Societies and the researches of their predecessors; or, if they had prospered, might pay in like manner a part of their debt to the Profession.

Cannot the Society take a more active part in making this great idea bear more fruit, both in increment to this fund and in adding to data for the practical engineer? Would it not show a finer spirit of appreciation toward those who build up this Foundation fund, if every dollar of its income from endowment was devoted to its main purposes, while the four National Societies took on themselves the whole expense of its administration?

On the second topic, "Professional Improvement", and on the third, "Encouragement of Intercourse", does not our present opportunity for doing more lie chiefly in a stronger and better organized support for each of the Local Sections? For the Board of Direction simply to frame some permissive rules for the Sections and then to give back $1 per year of the $20, or more, of dues collected, is not enough attention by the Parent Society. The Student Chapters also may well be given much more attention, and a strong effort put into the active development of the newly provided for Professional Sections within the Parent Society.

On this topic of intercourse, I take this occasion to express my regret at my inability to accept many of the hospitable invitations to address Local Sections and Student Chapters. I have gladly done so, as far as practicable, but with 34 Local Sections and 45 Student Chapters, it is obvious that some new means of showing proper attention from the home office to these scattered, distant bodies should be devised.

These second and third ideals of the Constitution—Professional Improvement and Intercourse—could be better satisfied by a larger membership. In his

Presidential Address,* eight years ago, Hunter McDonald, Past-President, Am. Soc. C. E., stated that the engineering development of the United States was then such that outside our Society there were three or four times as many qualified by the standards for membership, as there were inside the Society. Most of us who have broad contacts with what is going on in the United States, will on reflection agree with him, and agree that all three of the first aims of the Constitution, "Advancement of Science", "Professional Improvement of Members", and "Intercourse among Engineers", will be better satisfied when more of those now outside are quietly made to see that they should be inside the Society; not so much for what each can receive, as for what all can give in the maintenance of professional ideals.

By no means should the requirements for admission be lowered or less rigidly observed. The Constitution and By-laws make plain the intention to restrict membership to the specially educated, skilled, and supervising class and to young men who are educating themselves for high responsibilities. We owe it to ourselves and to the public to maintain Engineering as one of the learned professions.

All four of the objectives of the Constitution of this Society can be promoted through continually improving the quality of the publications. Civil Engineering presents the broadest field and opportunity of any branch of engineering for valuable papers and discussion, because of the scale of work, the variety of its field, and its many and broad public relations. A large proportion of the membership is in the direct service of the public, as City engineers, State engineers, public health administrators, railway administrators, and in various other branches of public service. Therefore, it is not strange that, in the course of the present year (1922), the Society is publishing a greater number of technical papers and more square inches of discussion of technical papers than any one of the other three National Societies (although less than the National Society of Chemical Engineers and Chemists which I hope will some time be in the United Engineering Society group).

While thus fulfilling the specifications of the Constitution, does not our monthly publication now partake too exclusively of the character of a cold storage warehouse, filled with valuable material, to some of which each member will certainly want to refer in the future, but not all of which is particularly interesting just now, particularly when one is a thousand miles away from the Engineers' Club.

The quality and the tonnage now going into this storage warehouse are admirable, but cannot we work in between these valuable papers more pages of immediate human interest to the junior members and to the thousands of distant members who pay their dues, 90% of whom can neither attend an Annual Convention nor enter Society Headquarters in New York City.

Cannot the *Proceedings* (or preferably perhaps an intermediate journal) carry more pages about what is going on just now in the engineering world, more of its inspiring story of current general scientific research, better book reviews by eminent practitioners, or teachers, which will give the young engineer a better guide to what he should add to his working library, out of

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1737.

his modest salary? Would it not be well to let each issue carry a few small half-tones and biographical sketches of engineers prominent in notable works of the day, thus adding more of the touch of human interest, and should not we find means for sending out better abstracts of current technical literature?

If the present ten issues per year become too bulky, split the material into twelve or twenty-six issues. Due regard for the distant member suggests that the issue should not be suspended two months in summer.

Such improvements, as suggested, cannot all be made at once, and to live up to present-day opportunities and to provide the right personnel for additional work will require more money than is carried in the present budget. How to obtain this money is another pressing and important question.

I will now proceed to the second topic, Hydraulic Science, and present some notes on its development and some suggestions of further development.

Old books on Science are always interesting and suggestive. I have found this true of Hydraulic Science and as I have taken books from the shelves of ancient libraries, I have found that although the art of the Hydraulic Engineer is old, his science is young, and some parts of it now appear to be in a state of arrested development. If this development is lagging, let us join in starting something!

Progress in Hydraulic Science

"Irrigation is the oldest branch of Applied Science in the world,"* says Sir William Willcocks, famed for hydraulic works and studies in Egypt and the Holy Land. One of the most remarkable pieces of ancient art that has been preserved in all the world, is a head, beautifully sculptured in obsidian, of the Egyptian king who completed Lake Moeris—greater than any irrigation reservoir since built—more than 4 000 years ago.

Thus, when the Mechanical Engineer, in pride of ancient lineage, points to Tubal-Cain or to the Artificer in Iron whom the wise Solomon seated at his throne, the Hydraulic Engineer may point to the important work of his ancestor who had charge of the canals in the Garden of Eden. For Sir William also says: "The events recorded in the early chapters of Genesis had their origin in a rainless land where life depended upon irrigation." He quotes from the second chapter of Genesis: "Out of Eden came a river which watered a garden", and says that, in the course of his explorations, he has discovered the precise spot. Also, he tells us how these rivers in the Holy Land have eroded their beds and impaired their possibilities for service since Biblical days.*

Speaking precisely, it was only an Art and not a Science by which, in those early days, water was regulated as it flowed down hill or was guided as it sought its level. The arts of controlling water were pre-historic, but Hydraulic Science did not have its birth until thousands of years after the dawn of history, or until about 300 years ago. Its birthplace was along the rivers of Italy.

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* "From the Garden of Eden to the Crossing of Jordan", by Sir William Willcocks, Spon & Chamberlain, New York.

Until very recently, all the Constructive Arts have been far ahead of their Sciences, as we now differentiate the terms, Art and Science, and most of the inventors and great builders of the past have not been scientists. The special mission of Science—the organization of knowledge, the discovery of natural law, and its expressions in formulas—has been to bring safety and economy into the Constructive Arts, and only very recently, and mainly in the new fields of Chemistry and Electricity, have Science and Mathematics yet accomplished very much in pointing the way to progress in the arts.

Very curiously, the old Roman, with all his skill in architecture and aqueduct building, seems to have had no conception of measuring velocity as a factor in quantity of flowing water, because he had no instrument with which to count seconds; therefore, he measured the discharge of his aqueducts in "quinaria", a sort of ancient "miner's inch",* of double the California quantity, determined by the diameter of a short outlet tube. His state of mind was much the same as that of the practical man of to-day who says, "as much water as a 6-in. pipe will deliver", or that of the country lawyers who drafted certain old water-power leases in which the water right was measured by the diameter of the penstock.

More than 250 years of slow earnest painstaking work by the greatest engineers and scientists of the Middle Ages was required to work out the simple fundamental formula, $V = \sqrt{2\,g\,h}$. The writings of Vitruvius, Frontinus, and even those of that greatest of all men of inventive genius, Leonardo da Vinci, reflect few, if any, glimmerings of exact science in the laws of flowing water, and it was not until about 300 years ago that a pupil of Galileo worked out his formula that the discharge from an orifice varied as the square root of the head. It was not until about 125 years later, in 1738, that the mathematician, Bernoulli, put the gravity factor into this formula and gave engineers the present fundamental hydraulic formula of $V = \sqrt{2\,g\,h}$, with which Hydraulic Science really began.

Tennyson says: "Science moves but slowly, slowly; creeping on from point to point", and Hydraulic Science moved very slowly and almost not at all until about 200 years ago, when there came a beginning of precise observation and experiment, and a development of Hydraulic Science, in both Italy and France. The fundamental formula for the flow of water in canals, aqueducts, and pipes, that is in general use to-day (the Chezy formula), was developed 150 years ago, at the water-works of Versailles; and, about 100 years ago, the French engineers had developed the mathematics of hydraulics to nearly the point where they stand to-day. In the next 50 years, there was much progress, but this has since lagged, and it is now high time we gave Hydraulic Science a new impetus.

Eighty-three years ago, fresh from his studies at the French School of Bridges and Roads, the late Charles S. Storrow, Hon. M. Am. Soc. C. E., of Boston, Mass., the best educated American engineer of his generation, wrote "A Treatise on Water-Works" which at that time was by far the best in the English language and was so clear, so complete, and so condensed, that it might serve as

* A "quinaria" was probably equivalent to about 2 miner's inches, or to 5 000 to 6 000 U. S. gal. per 24 hours.

a college textbook to-day and lead no one into serious error by its methods of computing flow through orifices and pipes or over weirs. His introductory chapter contains one of the best brief historical reviews of the development of Hydraulic Science that has yet been written, and his earnest plea—that engineers in charge of hydraulic work add to experimental knowledge—continues worthy of repetition to-day.

It was from 60 to 70 years ago that Francis made the greater part of his famous Lowell hydraulic experiments and published his book thereon, which is a classic in clear description and in detailing precision of measurement. He was the first in the world to design his experimental hydraulic apparatus of a size that could deal with full-sized specimens. Until his time, the orifices, weirs, and conduits used by the scientists to develop and prove their formulas, were but very few inches in diameter, and the fact that they obtained a fair degree of accuracy should give confidence in the laws of hydraulic similitude when we seek the laws of river flow in a laboratory. Francis' formula for discharge over weirs, his methods of testing turbines, and his methods for measurements of the discharge in canals by floats, remain standards for precise work to-day.

It was from 60 to 70 years ago that Humphreys and Abbot wrote their great book on the "Physics and Hydraulics of the Mississippi."

Although the past half century has been a period of marvellous development for all kinds of structures within the field of Hydraulic Engineering, and dams, aqueducts, municipal supplies, power developments, reclamation works, and canals have reached really wonderful dimensions in many parts of the world, particularly in America, experiments and systematic observations for increasing knowledge in important branches of the Science of Hydraulics have lagged behind the development of the art.* In the important field of training rivers, erosion of river beds, and in laws governing the deposition and transportation of sediments as affecting problems of relief from floods† this branch of Science has mostly slumbered, except at one or two small laboratories in Germany, of which I will speak later, although Fargue, in France, and Engels, in Germany, and a few others, have worked diligently to formulate observations on the fixing of locations of gravel bars on rivers of the European types, for the benefit of navigation. For example, the six rules of Fargue chiefly relate to stabilizing a navigation channel, to keeping the shoals so placed that the pilot will know where they are.

Most of the studies and books on river training have had navigation as their aim, while in these later years protection against floods and reclamation of fertile lands have become of relatively greater importance.

* Most of the college textbooks on Hydraulics fail to show that even the root given in the fundamental Chezy formula is probably wrong and should be written as $\frac{2}{3}$ or $\frac{3}{4}$ instead of $\frac{1}{2}$, the correction being now made in the variable coefficient. They give little warning that ordinary types of current meters in disturbed water may show velocities far from the truth. Their back-water formulas are not readily applicable to real rivers. They give no intimation of the twisting currents often found in circular penstocks and fail to note that the curve of distribution of velocity does not follow the parabolic law near the conduit wall in pipes or in river beds, but is greatly retarded. They say almost nothing about precautions for accuracy in application of formulas, etc., etc.

† The erosion and transportation of sediment probably has been studied more by English engineers on the great irrigation canals in India than anywhere else in the world, but their knowledge appears far from being yet organized into Science. A good résumé may be found in Buckley's "Irrigation Pocket Book" (Spon & Chamberlain, 1920).

What we now chiefly need is to find out how to control the sediments and improve the carrying capacity of rivers to the sea, so that with all possible speed and facility the main trunk will take away the water delivered into it by the branches.

We need to find out how best to make a river dig its bed deeper, instead of obstructing itself by rolling up its gravel bars into dams at every "cross-over" in the early stages of a flood. The late Professor J. B. Johnson, M. Am. Soc. C. E., stated the case well in 1884, in the *Proceedings* of the American Association for the Advancement of Science, in the language of a former Secretary of the Mississippi River Commission: "The first work of a flood is to impede its own discharge and the impediment outlasts the flood." Johnson cited observations at Columbus, Ky., comparing heights on the river gauge when the river was rising, with heights for the same number of cubic feet per second (1 100 000) on a falling stage, showing an average increase of 2.1 ft., and a total increase in flood height of 5.5 ft., due to gravel rolled up on to the bars.

This stagnation and need for a new departure in hydraulic experimentation and science was forced upon my attention, a few years ago, when I was reviewing the literature of river training in preparation for my studies in China, and has been brought to my attention during a recent tour of inspection, first along the Missouri River, near Omaha, Nebr., to inspect those "retards"* shown by Roy N. Towl, M. Am. Soc. C. E., in moving pictures at the Dayton meeting of this Society on April 5th, 1922; which "retards" are permeable elastic spur-dikes, built of trees, for training the river to avoid bridge abutments and to turn it away from eroding valuable farm lands; and it was still more forcibly impressed upon me only a few weeks ago when I inspected the threatened and the flooded regions along the Lower Mississippi at many points from Memphis, Tenn., to below New Orleans, La., including the crevasses near Ferriday and Poydras, the near crevasse at Tunica, and the threatening of a crevasse at Stanton, La.

Since that inspection, I have been reading the story of the Mississippi levees and jetties and proposed spillways in the library of river and harbor literature collected by the late Elmer L. Corthell, Past-President, Am. Soc. C. E., and as I have learned more details of the many controversies and of the diametrically opposite opinions of high authorities about river training, protection against floods, and about maintaining channels for navigation, which controversies have centered around the outlet of the Mississippi and certain harbors of the Gulf States, and the divergent views that prevail to-day in New Orleans about safe-guarding the future of the city, both as to floods and as to navigation, I have become more and more impressed with the idea that this branch of Hydraulic Science in particular is still in the stage of argument and opinion, rather than that of precise observation and determination of facts; and that the science of current control, particularly in rivers carrying large loads of sediment and flowing over beds of deep alluvium,

* A brief description of these retards is given in the paper by Mr. Towl, entitled, "Missouri River Bank Protection at Omaha, Nebraska", p. 1482.

is now very much where chemistry was before the universal use of the balance, or where electricity stood prior to the centimeter-gramme-second units.

On the Lower Mississippi, they have just been having the highest water ever known, and no one can appreciate the frightful conditions of threatened and ruined homes, of tens of thousands of square miles of the most fertile land in the United States worth, literally, hundreds of millions of dollars, threatened by inundation and by the loss of a year's crops, until he has been on the ground during high water and has seen this mighty river, ½ mile wide, 100 ft. deep, 50 ft. above its normal stage, rushing along with its surface within 6 in. of the top of thin soft earth dikes, from 12 to 18 ft. high, and here and there threatening to cut through them by a new eddy or swirl of the current; or until he has seen the temporary protective works hastily built by thousands whose homes and properties were threatened.

There are two distinct problems along the many miles of levees: One, that of designing and building a strong impervious dike to a proper height; the other, that of guiding and controlling the direction of the swift current so that it will not undermine and cut into and rupture the dike or "levee". One who is rash enough to risk an encounter with the authorities, American, French, and German, many venture to ask if there is not a third problem, namely, that of training the river to dig its bed deeper; and, in New Orleans, substantial citizens are asking that a fourth problem, many years old, that of spillways through the levees, be again considered.

Revetment of all the hundreds of miles of threatened banks against possibility of erosion is hopelessly beyond limits of expenditure. Advances in both the Art and the Science are needed.

For ten years past, I have been thinking of the benefits that might come from a hydraulic laboratory, built on a large scale, in which sundry important observations could be made, and nine years ago, I visited the new Flussbau Laboratorium of the Technishe Hochschule at Dresden, Germany. Three years ago, I urged the value of such a laboratory on a group of members of the Society gathered at lunch in San Francisco, Calif., urging that, if established at their great University of California, it would contribute greatly to the economic solution of some of the problems of the Sacramento River and of the problems presented by some of the torrential streams that at times rush down the Pacific delta cones.

On several occasions, I have suggested the value of such a laboratory *somewhere* in the United States, and what I have seen and read during the past few weeks has led me to believe that *now is the time* to urge the importance of immediately constructing a National Hydraulic Laboratory, on a larger scale, in some locality where it will be in a scientific atmosphere (for, in addition to many simple matters of experiment and observation, there are some very obscure phenomena of intricate hydrodynamics, colloidal physics, and other abstruse matters to be considered), which shall be at the service of whatever branch of the Government that may need it. First, for a season, say, in the service of the River and Harbor Engineers; next, perhaps, of the Hydrographic Branch of the U. S. Geological Survey; next, possibly, for some months, in the hands of the U. S. Reclamation Service; and, next,

perhaps, serving the Department of Agriculture; and sometimes serving a special purpose outside the Government service. Such a laboratory, operated in close parallel to studies on the real river, can be made to give a new impetus to several extremely important branches of Hydraulic Science, and give precise data which we lack in important fields. More about an hydraulic laboratory will be said later.

Resuming the story of progress, the Outlines of History of Hydraulic Science must be greatly condensed in an address of this kind, because to tell all the development that is of interest to an hydraulic engineer, would require some hundreds of pages.

Commonly, we forget how young all engineering science is. In most branches, it began its rapid development only about 100 years ago, or at most, 150 years, and followed close upon the greatest of all of man's inventions, the manufacture of power by the steam engine. It is hard for me to realize that my own engineering experience of 50 years covers one-third (one almost might say one-half) of the whole history of these great engineering activities, and that I knew personally the chief engineer of the earliest railroad in New England and the author of the earliest American book on Hydraulics, and that the greatest public work constructed by the National Government up to 90 years ago was the stone dry dock at the Charlestown Navy Yard, built from designs of Mr. Loammi Baldwin.

Although the Science is young, its Arts are old. The art of irrigation was highly developed in pre-historic times in many lands, in Egypt, Chaldea, India, China, and other parts of the earth. At the dawn of history, the flood waters of the Nile were regulated and stored for irrigation in Lake Moeris—far larger than any present-day storage reservoir—into which the waters of the Nile were led by the Canal of Joseph. In Szechuan, China, the present irrigation channels, watering many thousands of acres, are said to have been admirably designed and regulated, some thousands of years ago, by an engineer who worked out matters in great detail, set monuments for regulating the depth to which water might flow, and prescribed good rules of operation that are said to be still faithfully observed.

The arts of river control and canalization also received attention in those ancient days. Back in the half legendary days, about 4 000 years ago, China's most famous hydraulic engineer, the great Yu, is said to have successfully regulated rivers by dikes, guiding them into new channels, so that the people got along fairly well for 1 000 years, while his rules were followed. Many temples were built in his honor. The Grand Canal of China was built in part 2 400 years ago.

The Suez Canal retraces a small canal of many hundred years ago. A canal was dug from the Nile to the Red Sea in the reign of one of the Pharaohs, more than 1 450 years before the Christian era. The "fresh-water canal" used by the French engineers of 1863 to convey fresh water from Cairo to Suez had been in use for 2 500 years. Along the Tigris and Euphrates, large canals were in use for irrigation and for boats, 350 years before the Christian era, the ruins of which show them to have required much engineering skill.

In Greece, at Corinth, the American School of Archæology has excavated the remains of ancient fountains and conduits, which, as I inspected them, gave me much respect for the skill of the workmen of 2 000 years ago; but it was art and architecture that were in evidence rather than engineering design. In many places in the Old World, one finds remains of wonderful aqueducts, tunnels, and foundations built by the Romans about 2 000 years ago; and to me the most beautiful of all their structures, more beautiful than temple or palace, amphitheatre or forum, is that aqueduct bridge, the Pont du Garde, near Nismes, France.

Vitruvius, in his ten books on Architecture, written nearly 2 000 years ago, devotes the eighth book to Water Supply, but almost the only engineering data in its many pages of forgotten lore are some directions for constructing an accurate leveling instrument, the rule that aqueducts should not slope at a less gradient than 1 part vertical to 4 800 in length, or slightly less than 1 ft. per mile, and rules for the weights of lead water pipes. Caution is given about providing air vents and suggestions for laying clay water pipe with tongued ends, perhaps somewhat like that now made near Akron, Ohio. Vitruvius was wise in his sanitary precautions and strongly warns against poisoning from water conveyed in lead pipes, giving preference to pipes of clay; but in spite of its author's manifest effort to tell all that was worth knowing, it is really surprising to see how little this eighth book of Vitruvius contains of what we would call scientific information.

About 250 B. C., Archimedes, the great Greek scientist of ancient Syracuse, invented the screw pump, discoursed on principles of flotation of solids, detected spurious gold by determining its specific gravity in water, and is said to have discovered fundamental principles about the flow of water, but just what these were, I have not found recorded. Vitruvius quotes him as knowing that the earth was round and that a broad water surface had a curvature corresponding to its radius.

Julius Frontinus, Water Commissioner of Rome, about 70 A. D., left in his writings an admirable account of the aqueducts of Rome and their management. This account has been made available in English by Clemens Herschel, Past-President Am. Soc. C. E., in his excellent volume,* in which the translation of the original treatise is supplemented with a great deal of Mr. Herschel's own analysis of the ancient records, by a brief history of the development of early Hydraulic Science, and by many illustrations of appliances used by the Romans in distributing their domestic water supply. Frontinus seems to have been a faithful administrator rather than an engineer, but he compiled many notes and records, and had there been extant in his days any Hydraulic Science, beyond the simple facts that water would flow at about the right speed down an aqueduct having a declivity of about 1 ft. or 2 ft. per mile, and that it could be metered in a common-sense way by the area of the stream, or by the area of pipes leading out from the aqueduct, he would have told us all about it.

Outside the books of Vitruvius and Frontinus, as far as I have been able to learn, hardly anything can be found written on Hydraulics until about 425 years ago, when Leonardo da Vinci, engineer, artist, architect, poet,

* "Frontinus and His II Books on the Water Supply of the City of Rome, A. D. 97."

sculptor, and foremost of experimenters—greatest genius of all time—wrote on the "Motion and Measurement of Water" and invented, or first constructed, the navigation lock, which gave a new impetus to canal building. Some years ago, I visited the site of this first lock, as one visits a shrine. Although water still runs down the canal, it now serves to generate electricity for Milan.

With all his genius and his art, and his wonderful skill in mechanical invention, Leonardo did not add so very much to Hydraulic Science, and, after his time, it seems to have slumbered for 100 years, until the days of Galileo and his pupils, Torricelli and Castelli, each of whom was a real scientist and began the development of precision of measurement and the systematic arrangement of facts expressed by mathematical laws.

The experiments of Galileo on efflux appear to have been begun in efforts to construct a water-clock, measuring time by the flow of water through an orifice, which should be a more accurate timepiece.

It was Castelli, working under the direction of Pope Urban VIII, in 1628, who appears to have first published a treatise on rivers, and he appears to have been also the first to introduce velocity as a factor in measuring the discharge of rivers; but he supposed wrongly that discharge varied *directly* as the head.

In 1643, Torricelli discovered that water issuing from an orifice followed the law of falling bodies, on which his master, Galileo, had experimented at the Tower of Pisa; but although he got the idea of "the square root of h" all right, it was not until nearly 100 years later that the factor "$2g$", was put into the equation by mathematicians. Torricelli appears to have been the first who argued that the acceleration of rivers was due to the slope down which they ran.

Forty years later than Torricelli, in 1684, the French physicist, Mariotte, gave a new impetus to Hydraulics by his observations on flow from tubes and orifices, and he also wrote a treatise on the movement of water.

In Italy, there was great need to study means of relief from the floods of torrential mountain streams that brought not only floods, but also débris and sediment that clogged the river beds and added to the woes of the people, particularly along the lowlands of the River Po. Therefore, it is not strange that river science had its birth in Italy.

In 1697, Guglielmini published the first part of an elaborate work, "Della Natura de Fiumi", the second part of which was published in 1712, after his death. He appears to have been one of the earliest scientists to grasp fundamental principles and made many contributions to Hydraulic Science, probably more than all the others who had preceded him, but he went sadly astray about a few of them, for example, in his notion of the distribution of velocity at various depths. His work is spoken of very kindly by authors of a century later. Undoubtedly, he was the pioneer in River Hydraulics and the old quotations from his works show that he started several important questions that are not settled to-day, and to answer which we need a National Hydraulic Laboratory.

Mathematical science by this time was well expressed in algebraic equations, also the application of Calculus to Science was receiving great development, and the great mathematicians seem to have been fascinated with the problems of

flowing water, but made a bad mess of trying to devise their laws from *a priori* reasoning instead of by observation in field or laboratory. Sir Isaac Newton, the greatest of all these mathematicians, tried his hand at formulating laws about flowing water, but found no great success in this field, although he developed some beautiful equations in the "Principia", published about 1714. Soon after 1714, there came a rapid succession of authors on Hydraulics.

In 1718, Marcus Paulini experimented on the discharge through orifices and discovered that it could be increased by adding a short tube. Frontinus had previously stated that this principle was known in Rome, and that thrifty Roman water takers had applied it to increasing the discharge from the aqueduct into their premises by adding an enlarged tube to the official nozzle.

In 1730, Pitot, member of the French Academy of Science, invented the Pitot tube which in more elaborate form we use to-day, for measuring velocity and disproved Guglielmini's ideas on the distribution of river velocities at various depths, although his precision of measurement was crude.

In 1732, experiments made on the flow of water in the pipes leading to the Versailles Fountains, were reported to the French Academy.

In 1738, Daniel Bernoulli, and, in 1742, John Bernoulli, father and son, began the establishment of sound mathematical theory as a basis of Hydraulic Science and developed the doctrine of living force.

In 1743-52, d'Alembert, another great mathematical physicist, made important contributions.

In 1764, Paul Frisi, Professor of Mathematics at the University of Milan, wrote his celebrated treatise on the "Nature of Torrents", quoting largely from Guglielmini's observations and giving proof in many ways of being an earnest seeker after truth. Other Italian writers on the flow of water flourished about this time, their interest being aroused by the flood problems of the Po and other Italian rivers. In 1765, Lechhi, of Milan, also wrote an elaborate treatise.

In 1768-71, Euler, the great mathematician, published, at St. Petersburg, a treatise in which he attacked the problem of the flow of water and made useful mathematical contributions.

Up to about this time, or as recently as 150 years ago, the sum total of contributions to theory, other than the one formula for efflux, $V = \sqrt{2gh}$, was of little practical importance, because of the lack of care to test theory by experiment and from the lack of precision in measurement; but from about 1764 to 1774, the experimental era was inaugurated, and both in Italy and in France the hydraulicians had reached the conclusion that formulas must come from experiment and not from pure mathematics. Within the following hundred years, the science was developed nearly to where it stands to-day.

In 1775, Chezy, as already stated, gave us his extremely valuable formula for the flow in conduits, but without much discrimination as to the effect of roughness of the wetted surface.

In 1782, Belidor wrote a monumental work on hydraulic architecture, copies of which I have found in three of the early American engineering libraries.

In 1784, the Academy at Toulouse, France, reported experiments on the discharge of orifices and some notes on the junction and separation of rivers.

In 1779-86, Dubuat reported his results of ten years' reports and experiments. Some of his data on the transportation of débris are still reported in textbooks of to-day, as if his data were of practical value—which they are not, because of having been obtained by experiments on the effect of a current of water in moving sand and pebbles lying loosely on the smooth bottom of a little wooden trough only 18 in. wide and less than 1 ft. deep.

I would not leave the impression that I have laboriously read the works of all of the previously named authors, but from time to time I have turned the pages of many of them and have found much of interest in tracing parts of this history in certain old libraries, particularly, in the Locks and Canals Library at Lowell, collected mostly about 75 years ago by the late James B. Francis, Past-President, Am. Soc. C. E., and in the remarkable library of Mr. Loammi Baldwin, who has been called "The Father of Civil Engineering in America", the collection of which was begun about 100 years ago. Baldwin's library was doubtless the best possessed by any engineer in America in his day, and it is extremely interesting as illustrating the breadth of culture of this early engineer. It is particularly rich in the French hydraulic works of 100 years ago, and contains also a few books in the Italian language, notably a treatise, in three thick volumes, on the motion of water, published at Parma in 1766, the second volume of which is entitled "Della Natura d'Fiumi: Trattato Fisico Matematico dell Dottore Dominico Guglielmini", etc.

The Baldwin Library also contains a copy of "Nouveaux Principes d'Hydraulique Appliqués à tous Objets d'Utilité et particulièrement aux Rivières", published in 1687. It is interesting to note this author's good words for Guglielmini, the author on river hydraulics of nearly a century before. On page 3 of its Introduction, this book contains a very interesting reference to Galileo's opposition to straightening the Bisenzio, and his seven principles, derived *a priori,* in opposition to Bartolotti, who proposed straightening the river. The author states that "Galileo had the misfortune of making his opinion prevail in spite of the truth." (This remark reads like some of those made 235 years later in the controversies about the Mississippi.)

The Baldwin Library contains also a copy of the work, "Recherches sur la Construction la plus Avantageuse des Digues", a prize essay published by the Toulouse Academy, written by Citoyens Boussut et Viallet in 1772, which, in certain of its diagrams, illustrates that groynes and retards for the protection of river shores had been used more than 150 years ago.

In this library is also to be found a copy of De Prony's work, of 1822, on the Pontine Marshes, which he had studied as member of a commission in 1810, as well as a copy of "Essai sur la Théorie des Torrens et des Rivières, par Citoyen Fabre, Ingénieur en Chef des Ponts et Chaussées", published in 1817; also a book of 280 pages, entitled "Les Moyens les plus Simples d'en Empêcher les Ravages, d'en Retrecir le Lit et d'en Faciliter la Navigation" ("The Most Simple Means of Restraining the Ravages of Rivers, of Narrowing or Straightening the Bed, and of Facilitating Navigation").

The library also contains the work of MM. Ponclet et Lesbros on the discharge of orifices, published in 1832. It contains many other books of interest to an engineer, showing the state of these arts and sciences 100 years ago.

Incidentally, it may be of interest to remark, as showing the broad culture of "The Father of Civil Engineering in America", that in addition to many scientific works in English and French, and a few in Italian, Mr. Baldwin's library was rich in classic literature and in works on Natural History, for which apparently he spared no expense, for we find here an original set of Audubon's celebrated volumes, "Birds of America", with hand-colored plates, which was published at $1 000 per copy. The scientific part of the Baldwin Library is now at the Massachusetts Institute of Technology, but the parts relating to general literature and Natural History are still in the old Colonial mansion of the Baldwin family at North Woburn, Mass.

A copy of the celebrated treatise of Paul Frisi, on "Torrents", may be found in the Library of Brown University. This English translation is of interest, having been made more than 100 years ago by an English engineer officer for the use of the English engineers in India in their early days of developing the vast Indian systems of irrigation. Few in America appreciate the present magnitude of those Indian irrigation works or the early day at which England began this beneficent work in India. This conscientious engineer-translator visited Italy to check up the general accuracy of the statements in the original work. The book was dedicated to Warren Hastings. It is a beautiful sample of the printer's art of 100 years ago, contains many references to the investigators who had gone before, and raises several important questions about which there is still controversy, such, for example, as the raising of river beds by diking their floods. There are chapters on the subsidence of the sea coast, and on deposits of sediment, and experimental proof that gravel stones could not be ground down into sand during their passage down the whole length of a river, wherefore those brought in by mountain torrents must accumulate and raise the bed of the river near the foot of its steeper slopes; and on the raising of certain up-stream sections of the beds of Italian rivers within the historic period, he presents much proof. He also shows that the débris dams of California were anticipated several hundred years by those of Italy, which were not always successful, etc. All this shows Paul Frisi to have been such an earnest seeker for scientific truth that we wonder he could not find more of it. This book of 168 years ago details controversies between the highest authorities over the flood problem of the Po and other Italian rivers, which very much resemble those that have raged up and down the Lower Mississippi and the Sacramento and, thence, have spread to Washington, during the past 50 years, and are still unsettled. They were then, 168 years ago, violently discussing the efficacy of cutting spillways in the dikes to take off their floods, somewhat as at New Orleans to-day.

From the accounts of Pliny and Tacitus, they seem to have been discussing and trying cut the same ideas in Rome (with poor success), 1 600 years before. Frisi quotes Tacitus as recording a dictum of the Roman Senate concerning straightening the Tiber, "Nature has known how to provide for our wants

much better than Art in assigning to rivers those courses, boundaries, and limits, which are most apposite". This reminds one of those recent authors who oppose training rivers to straight courses "because Nature always guides them on curves", without reflecting that Nature's purpose was largely that of delta-building, while Man's is flood protection or navigation.

As an engineering treatise, this book of Frisi is chiefly interesting in showing the state of the art at that time and the small amount of accurate Hydraulic Science that had been made sure of 100 years ago. Nevertheless, it is a wonderfully interesting book and raises many questions, and shows that its author had traveled widely and studied diligently. It illustrates better than any other old treatise on Hydraulics, the slow process of organizing knowledge into Science.

It is interesting to read Frisi's criticism of the absurdities into which mathematicians, even the great Sir Isaac Newton, had been led by efforts to develop laws *a priori* from mathematical reasoning, and Frisi's declaration that Hydraulics is a branch of physics rather than of mathematics, and that he especially renounces the hypothetical calculations of his predecessors. Nevertheless, like much that has been written on river science down to the present year, 1922, and is still current, Frisi's own writings are largely flavored with speculative philosophy.

The first clearly written and concise treatise which I have found anywhere is that by the late Mr. Charles S. Storrow, previously mentioned, which was written 83 years ago.

Bennet's translation of D'Aubuisson's "Hydraulics" was another good American book of many years ago, written, I have been told, by a man of exceptionally fine character, mostly at home in the evenings and late at night, while he was trying to cheer up an invalid wife by faithful companionship.

An admirably full bibliography tracing the development of the hydraulic theory of flow in rivers from the earliest times down to 62 years ago, with an appreciation of the contributions to the science by each author, can be found in Chapter III, pages 187 to 228, of Humphreys' and Abbot's "Physics and Hydraulics of the Mississippi." Perhaps the most noteworthy contribution to river training since that time is that of Professor James Thomson on the travel of detritus diagonally across rivers at bends, reported to the Royal Society in 1877 (page 356), or 45 years ago.

It was 65 years ago that the publication of Henri Darcy's researches on flow of water in pipes called attention to the remarkable differences in loss of head, or slope, caused by different degrees of smoothness of the walls of pipes, and developed the parabolic law of distribution of velocities in pipes by means of the Pitot tube; but the remarkable series of experiments on open canals by Darcy and Bazin was not published until 1863, or two years subsequent to the publication of the investigations of Humphreys and Abbot.

These experiments on flow of water in straight pipes and straight open artificial canals, made in France at Government expense, by Darcy and Bazin, with thoughtfully designed apparatus and special attention to observing the effect of smoothness of conduit wall on resistance to flow, and measurement of the distribution of linear mean velocities, are fine examples of the service of

a well equipped laboratory to Science and the Constructive Arts. The scarcity of similar examples shows that research of this kind must be financed by the Government, or mostly remain undone. Bazin made subsequently some important contributions to data on weir flow, but the data of the epoch-making Darcy and Bazin laboratory researches have not been added to in the sixty years that have followed, to anything like the extent that one might expect, nor has the work so admirably begun by Humphreys and Abbot been followed up as the importance of the subject deserves.

Nearly 50 years ago, we had the excellent translation of Weisbach's Hydraulics (first published in German 76 years ago), by our fellow member the late Eckley B. Coxe, M. Am. Soc. C. E., and many admirable college textbooks on Hydraulics have been published since, especially notable among which are those of Professor Mansfield Merriman, M. Am. Soc. C. E., Professor Irving P. Church, Affiliate, Am. Soc. C. E., that of David A. Molitor, M. Am. Soc. C. E., giving refinement of formulas, Professor W. C. Unwin, Hon. M. Am. Soc. C. E., the late Professor Bovey, of Montreal and London, Hughes and Safford, and the recent book of Professor William H. Durand dealing with flow in pipes. Revy has described his researches on the great rivers of South America. Cunningham and Gordon, Kennedy and Buckley have written excellent books giving fragmentary observations on the rivers of India, and Thomas and Watt, United States Assistant Engineers of long experience, have published a monumental treatise on the improvement of rivers, dealing mainly with structural designs. I do not find, however, in any of these books the theories, or the data, that we need in training rivers for relief from flood; nor do I find it in the excellent work of Professor Van Ornum, formerly a U. S. Assistant Engineer, or in the works of those authors experienced in the special problems of India, Belasis, Strange, Buckley and Parker, although the two latter present some suggestive data from the Indian works. These latter manuals are intended for the practicing engineer rather than for the college student. Spring's "Training of Rivers on the Guide Bank System", also gives many fruitful suggestions, as does also the development of the "Bell Bund" guide dikes at several Indian bridges.

The reports of the several International Navigation Congresses of the past 20 years give many interesting opinions and some highly useful observations, but as I have read them, the impression has been vivid that they were concerned chiefly, as the title shows, with river training for navigation, not for flood relief. Although the preponderance of authority is for training rivers along the curving lines of Nature, I do not find that the engineers of the great irrigation canals of India which have to take care of some of the worst sediment-carrying waters found anywhere in the world, lay them out on other than the straightest practicable lines. Their problem (like our's in training rivers for flood relief), is the *conveyance of water,* not its navigation, or the fixation of shoals for the convenience of pilots.

In Germany, Professor Hubert Engels has published an admirable treatise dealing with river and harbor structures, of which a new edition is just out, which appears to give a résumé of all recent European writings on river training. In France, Fargue has written much of interest in trying to

formulate laws of river control; but all these treatises have had to deal with scant data, and many of the fundamental scientific laws governing erosion, transportation and deposition of sediments, and the art of river training, are not yet well established. Even Kennedy's laws and rules for silt control in canals and rivers, derived from many observations in India, while heartily approved by many experienced engineers, are questioned or doubted by others.

Three years ago, as previously stated, in preparation for my work in China, I arranged with Hardy Cross, Assoc. M. Am. Soc. C. E., then Assistant Professor of Civil Engineering in Brown University, to make a thorough search for everything in recent contributions to the science and art of river training, which seemed worthy of an abstract, in the principal libraries, beginning with the Corthell Library at Brown University and covering the Libraries of Harvard and the Massachusetts Institute of Technology, the Boston Public Library, New York Public Library, Engineering Societies Library, and the Library of Congress, also including the *Minutes of Proceedings* of the Institution of Civil Engineers and the *Transactions* of this Society, and of the several International Congresses for the improvement of navigation and many recent French and German publications. He compiled, with excellent judgment, about 1 000 typewritten pages and photostat copies of abstracts, classified them, and brought them into parallel for my use.

My general impression, born of weariness, after laboriously reviewing his many papers, was that we had collected mainly a mass of conflicting opinions, with a comparatively small amount of new data. This first impression was too severe, for much was found that is interesting and valuable, which I would like sometime to arrange more fully and publish as a volume of classified topical abstracts, containing perhaps 500 pages, for the convenience of other students and in the hope that the state of uncertainty thus revealed would stimulate a new departure in research in laboratory and river, that would lead to certainties instead of merely a collection of conflicting opinions.

There is promise of some noteworthy additions to the literature of river training in the near future. Col. C. McD. Townsend, U. S. A. (*Retired*), M. Am. Soc. C. E., for many years Chairman of the Mississippi River Commission, has ready for publication a new book, in which all who know of his many years of great interest in river and harbor problems, may hope to find much information of value.

J. A. Ockerson, Past-President, Am. Soc. C. E., reported* in the course of his recent address at Dayton, Ohio, on "Flood Problems of the Mississippi", that work is now in progress on the task assigned to a special committee of its members, "of preparing a full report of the work done under the Mississippi River Commission and the results attained, 'in order to bring together with a view to publication and make available the great mass of valuable data relating to river hydraulics, covering the physical investigations of the varying elements that make up regimen of the Mississippi River, as well as the instrumentalities that have been developed and tried out by the Mississippi River

* See p. 1477.

Commission and its assistants in channel improvement and flood control of the river'."

It is to be hoped this report will be published soon and that it will bring together the valuable views and opinions scattered through the reports issued annually for the past forty years by the Mississippi River Commission, and also all the important data from reports published from year to year as sections of the general report of the Chief of Engineers, U. S. Army, who is charged with all work performed on rivers and harbors under appropriations by Congress.

Although the Science of Hydraulics has made small progress in the past 50 years, the Arts of Hydraulic Construction have been wonderfully developed in many structures built on a stupendous scale.

The recent Catskill Aqueduct, in a single conduit, has nearly ten times the combined capacity of all the nine aqueducts of Ancient Rome. We now have no occasion to build such magnificent architectural structures as the Pont du Garde, because thanks to the progress in manufacture and the low cost of structural steel, we get far more capacity for less money in a 10-ft. steel siphon. The Los Angeles Aqueduct in boldness of conception far outranks anything of olden time. Moreover, it supplies to the community not only water, but from this water as it flows to the city, it provides electric power and light, which services were not dreamed of even a half century ago. The Hetch Hetchy Aqueduct, now in progress of construction, is on an even more stupendous scale. The Boston Aqueducts, although now outranked in size and length, should not be overlooked, for it was in their construction under the late Joseph P. Davis, M. Am. Soc. C. E., the late Alphonse Fteley, Frederic P. Stearns, and Desmond FitzGerald, all Past-Presidents, Am. Soc. C. E., that a school of scientific water-works construction was established in Boston, the influence of which has spread around the world; as has also the influence of Boston and Massachusetts in safeguarding the purity of domestic water supplies, under the devoted leadership of the late Hiram F. Mills, Hon. M. Am. Soc. C. E., who, shunning all publicity, gave the best that was in him to the service of his fellow men, without fee or salary, during the larger part of his working hours for thirty years.

In power development, also in turbines, the scale of construction has become truly wonderful. About twelve years ago, at Keokuk, Iowa, the Mississippi River Power Company had designed a single hydraulic turbine having a power capacity about two-thirds that of all the eighty turbines in Lawrence, Mass., or to all those in Manchester, N. H., and then placed fifteen of these huge units in a row. A few months later, even larger units were put into service at The Cedars, on the St. Lawrence, and now near Niagara, on the Canadian side, there have just been put into service, single units, each of which has a power capacity about equal to the sum total of the developed turbine capacity at Lowell, Lawrence, and Manchester, the three great power cities on the Merrimack, which were three of the greatest water-power centers in the world only a half a century ago. In these great modern water-power developments, the hydraulic turbine yielding more than 90% of the theoretic power in water fall, has been brought as near to perfection as can ever be hoped for.

Meanwhile, water-power development in icebound latitudes has been greatly helped as the outcome of a research in pure science, by Dr. Howard T. Barnes, Professor of Physics in McGill University, who has shown us how to conquer anchor-ice troubles by raising the temperature of the water less than 0.01° Fahr.

The art of building great dams, both of masonry and of earth, has been developed on the great municipal water supplies for Boston and New York City, and, particularly, for the U. S. Reclamation Service, until now it is proposed to build on the Colorado River, a concrete dam, having its crest 605 ft. above the present low-water surface, or higher than the Washington Monument. Its proposed height above bed-rock foundation is 745 ft., or about the same as the topmost pinnacle of the Woolworth Building above Broadway.* In recent years, several masonry dams have been built that are much taller than the Bunker Hill Monument, the granite shaft of which, in Daniel Webster's day, was considered a wonderfully tall masonry structure.

In irrigation works, the hydraulic engineers of the United States, particularly those attached to the U. S. Reclamation Service, under A. P. Davis, Past-President, Am. Soc. C. E., have developed works far beyond even the conception of the Ancients, although they have as yet hardly equalled the acreage brought under the service of water in India during the past century by English engineers.

The canal construction at Panama and the solution of its important hydraulic problems are so well known to all engineers that no especial mention of them is needed here.

Doubtless, I will be criticised for having intimated, in the face of all those stupendous works, that Hydraulic Science has been moving too slowly. In partial answer, it may be said again that it is not the art that has slumbered, but important parts of the science, notably river hydraulics and the means of training rivers to flow peacefully and to carry their load of sediment to the sea, and to dig themselves deeper, if the character of the bed and the general situation permit.

Some explanation for this lack of progress may be found in the fact that some of these American river problems are larger and more difficult than those in other parts of the world, and that it is the American habit to go ahead with the construction without waiting to develop the scientific theory to the utmost nicety. We can build good aqueducts and safe dams notwithstanding uncertainty about the root of h in the Chezy formula or the precise value of the coefficient of discharge for a round crested weir, and we can develop power without waiting to untangle all the vagaries of preliminary stream gaugings. Progress in construction is often more profitable than precision, and, for example, at Panama, the Federal Government was wealthy enough to stop the slides by simply digging away everything which it seemed might sometime slide, instead of taking the chances involved in delay while profound studies were being made on the increasing frictional stability of earth by the abstraction of water, or by other means. Nevertheless, in the long run, waste

* See "Problems of Imperial Valley," U. S. Senate Doc. 142, 67th Cong., Govt. Printing Office, 1922, Plate IV, following p. 21.

results from all such lack of knowledge, and this seems particularly true with regard to river training and protection from floods.

Many of these great works, just mentioned, have added valuable engineering data. Precise measurements giving the relations of flow, slope, and area, have been made in these great modern aqueducts. Messrs. Fteley and Stearns utilized the Boston Sudbury Aqueduct for their weir experiments. The late Hamilton Smith, Jr., M. Am. Soc. C. E., contributed experiments on pipes in California and on orifices at Holyoke, Mass. Herschel has given the results of experiments on flow in the great steel pipes of the East Jersey Water-Works, etc., and the U. S. Department of Agriculture has given us new and valuable data on the coefficients of flow in irrigation canals and flow in drain tile, which add more patches to that wonderful piece of hydraulic patchwork, the Kutter coefficient for the old Chezy formula. This formula with a coefficient, somewhat ragged after 53 years of wear, still covers fairly well most problems of the construction engineer, but it is high time we had something better.*

Moreover, within these recent years, the art of gauging flowing water has been much improved. Past-President Herschel, in his Venturi meter, transformed an ancient curiosity into a most valuable invention, and recently has partly developed and applied for a patent on a new gauging instrument of remarkable simplicity and promise in his new method of measuring the head and area by means of a perforated pipe on a round crested weir, and B. F. Groat, M. Am. Soc. C. E., has developed the art of Chemi-Hydrometry for which he has received the Norman Medal of the Society.† Meanwhile, the electric generator has given a far more convenient absorption dynamometer for measuring output in water-power turbine tests than the old Prony brake.

A National Hydraulic Laboratory

Early in this address, I mentioned the hydraulic laboratory at the Dresden Engineering School which, although small, has been helping to solve problems of river training, under the skillful management of Professor Hubert Engels, who has devoted his life mainly to studying and teaching Hydraulic Engineering and is the author of one of the best books yet written descriptive of the problems and works of the hydraulic construction engineer.‡

About 20 years ago, Professor Engels established a small laboratory for experimenting on models of rivers, worked out mathematical relations between small and large streams, and demonstrated that the model could give dependable results. In 1913, on the moving of the College to a new site, a new Flussbau Laboratorium was built on a larger scale. This new laboratory was still unduly limited by the space left for him by the architect

* Twenty-nine years ago, hoping to add something to Hydraulic Science, the speaker made elaborate experiments of wide range and great accuracy on loss of head corresponding to a wide range of velocities in all commercial varieties of iron water pipe, tees, and elbows, from ⅛ in. to 8 in. in diameter; also, on new, especially smooth brass pipe, from ½ in. to 4 in. in diameter; also on the loss of head in pipes given the roughest possible interior by lining them with expanded metal lath, and on a great variety of elbows, tees and curves of different radii. He greatly regrets that he has not yet found time to prepare these observations in proper shape for publication, but hopes to finish the reductions and drawings within the coming year. His own shortcomings in failure to publish, make him charitable toward others who might have helped the good cause.

† *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 951.

‡ "Handbuch des Wasserbaues", Leipzic, 1921.

in the basement of the new college building, and hardly a third part as wide or as long as the dimensions that I would suggest for the American National Hydraulic Laboratory. Nevertheless, by the aid of these small models, Professor Engels has worked out various practical problems for rivers and harbors in Germany. A similar laboratory has been built at the Civil Engineering School at Karlsruhe and still another at Berlin. Also, there have been small-scale models built in England and in France for studying river and harbor improvements, and these have been discussed by eminent authorities. After a partial examination of what has been written about them and a review of various discussions of the use of models in the study of river problems, this German type seems best suited for our present needs and promises results of great value *if properly used in parallel with studies on the real river.*

The principal apparatus in the new Dresden Laboratory comprises two canals, or tanks, each about 75 ft. long, the principal canal being about 6.5 ft. wide with an annex, or wing, giving a total of about three times the normal width, within which river bends can be modeled. One tank has a fixed support at one end while the other end rests on jack-screws by which the whole length can be set at various degrees of inclination.* Within this long, tilting tank, a model of the river channel to be experimented on is built up, with all its sinuosities and variations of cross-section. Sometimes the model river bed is built up of earth and sometimes it is made with a base of plaster of Paris, which either can be left bare or can be veneered with earth of the quality under experiment. Sometimes, in studying the deposit of sand-bars, in order to obtain greater visibility, pulverized coal is used to represent the sedimentary deposits, because of its special visibility through the muddy water against the white plaster-of-Paris background.

The same water is circulated over and over again down the model river channel and back through a pipe. By means of a centrifugal pump and by regulating the slope of the tank any desired rate of discharge can be obtained. There are elaborate devices for checking pulsations in the delivered flow, and the experiment can be made with various velocities. After the water has run under a given condition for a time, the model channel can be emptied, and the state of erosion and of deposition of sediment can be observed and photographed.

In Professor Engels' recent publications, the model is shown as arranged for experiments on the Rivers Weser, Rhine, and Danube, and illustrations are given of experiments preliminary to harbor improvements at Düsseldorf, Ymuiden, etc. Also, experiments are described on scour and deposition of sediment caused by various designs of spur-dikes or groynes, some pointing up stream, some down stream, and some squarely across, with photographs showing the deposition of the sand waves developed by the eddies around the ends of these groynes and in the adjacent channel bed. As far as they go, the results shown in his publications seem remarkably instructive, considering the small dimensions of his model rivers. He prefers a depth of water not exceeding about 8 in., because with greater depth of sediment-laden water, the motion of the sand grains along the bottom cannot be observed. I am led to believe that

* A recent publication shows the river tank set up on rigid supports.

more convincing results could be had from the larger tank that I would now propose and in which either small or large depths could be used.

I beg that I be not misunderstood as claiming or suggesting that the several problems of river training, and erosion and sand-bar deposition, can be completely solved in such a laboratory. Tests on the model stream, seldom of more than 2 ft. in depth or 10 ft. in width, would be suggestive rather than conclusive, and should be supplemented constantly by reference to the full-size specimen. The mathematical laws of correspondence seem to have been fairly well demonstrated in Professor Engels' laboratory, but by this frequent comparison they would soon become well established. It needs no extended argument to prove that variants in form of dike, jetty, or in slope of river bank, or seashore and bulkhead, and all the varying relations of velocity to erosion, transportation, and deposition of sand and gravel, could be tried out far more quickly and economically in the model, than on the river or shore, and that thereby the number of years through which the threatened communities must wait for relief could be shortened and the deepening of harbors hastened.

My conception of such a laboratory for America, is that it should be National in scope and available for the use of any one of the branches of the service that has problems of the kind that it could help solve. After serving for studies of river flow, coefficients of pipe flow, and crest discharge, it could be quickly refitted to help solve problems of shore protection against wave wash, like those of the New Jersey or the Santa Barbara shores, and when the series of experiments was completed, the apparatus could be maintained at very small expense until the next problem came.

It is not necessary to locate this National hydraulic laboratory on a flowing river or at a water-fall. There are certain advantages of regulation and precision of measurement which can be best secured by circulating the same water over and over again, by pumps of the centrifugal type. One main requirement is a convenient source of electric power, commonly about 50 h. p., seldom more than 100 h. p., for a few hours, intermittently. If the laboratory is further developed with apparatus for new determinations of accurate data for the discharge over large weirs and various forms of dam crests, up to 5 ft. in depth, there might be sometimes a brief call for 1 000 h. p. or more for a brief experiment that could be performed at night or at off-peak hours. The two canals and the main tanks should be housed for winter use and for protection from wind and rain.

In a preliminary and tentative way (but subject to revision after conference with others interested), I am proposing that the tilting tank for the model river be about 250 ft. in length, 20 ft. in width, and about 3 ft. in depth, with a broad wing giving a width of, perhaps, 50 ft. for laying out river bends, and that this tilting tank be fixed at one end at a permanent elevation on a fulcrum or pivot, while it is supported elsewhere by a series of jack-screws about 25 ft. apart, all actuated by worm wheels of varying pitch, proportional to the movement during inclination, so that all can be actuated from a single shaft and motor and the model tank quickly set at any desired gradient, from horizontal to, say, 3 per cent. After a run at one slope, the inclination can be changed in a moment's time by these screws without other-

wise disturbing the model. The same apparatus could be quickly adapted for tests on models of jetties for harbor entrances and for studies of shore protection.

Within this tank, a rough foundation of any convenient material could be shaped to represent a river channel, say, from 5 to 10 ft. wide by 2.5 ft. deep, or smaller, and this base veneered, or covered over, with earthy material from the river in question; or, sedimentary material of the kind carried by this particular river could be stirred into the water while one watched its collection into shoals or sand-bars, and the laws of deposition in eddies behind spur-dikes or retards could be studied. Also, one could study the law of re-distribution of sediments in the bed of a river during floods, by which, in many critical places, material is eroded during a flood from the pool near the concave shore and deposited along the bar, at the point of contraflexure.

One could also develop further information on the respective merits and disadvantages of straightening a channel that has many big bends, which topic has been under dispute since the time of Galileo, and on which I will say more later.

A large amount of much needed information could also be had on the best radius or curvature for the end of spur-dikes placed in series, for maintaining a straight-line channel. There can be no doubt that there is an optimum radius for such curvature and that a dike, with its end curved down stream on a long radius, can be formed so as to lessen the irregularities of the sand waves in the channel bottom and lessen the tendency to undercut the end of the spur-dike or groyne.

Obviously, it is far cheaper to begin the development of the best form of groyne in a small, cheap model than to build a groyne of full size in the river; moreover, the effect of changes in outline can be far more quickly studied as the experiments progress. One can perhaps cover as wide a range of experiment for finding which is the best form in 6 days on the model as he could cover in 6 months on the river. Nevertheless, studies on the small model and on the large river should go on almost simultaneously, and the distribution of velocities in eddies and the capricious forms of sand waves in river, and in model, must constantly be compared, in order that the tests on the small model may not lead one astray.

The admirable experiments on the transportation of sand and river débris, carried on by Professor Grove Karl Gilbert, of the U. S. Geological Survey, at the University of California, in 1914, could be extended.

The most difficult requirement of all in establishing a National Hydraulic Laboratory, is that of finding a director who shall have a clear head, a broad experience in the field, and a devotion to science like that of the late Professor Gilbert.

Last, but not least, is the matter of swirls and eddies and sundry obscure forms of vortex motion in the water, which some of us who have been observing watercourses for many years, believe are at the bottom of many of the problems of erosion and transportation of sediments. The swirls and eddies of the big river could be studied in parallel with studies in a laboratory of the dimensions proposed. I believe that by glass plates in the walls and powerful

illumination of a thin optical section by means analogous to those used with the ultra-microscope and with the use of bright sand grains and a high-speed motion picture camera, much could be learned about this vortex motion.

The tank in the laboratory of Professor Engels has been used for a variety of other experiments, for example, studies have been made relative to the shaping of jetties at harbor entrances on coast lines where great quantities of sand are swept along by the tidal currents and clog the entrance to the harbor; and studies also have been made of varients in the outline of these harbor jetties, for lessening the deposits, or of disposing of them so as to lessen their obstruction to navigation.

Other trials have been made in Professor Engels' laboratory on the effect of various forms of bulkhead or sea-wall in resisting the action of storm waves, and for obtaining a shape of the vertical cross-section of the embankment or sea-wall that will leave the sand carried by waves and littoral currents in good shape. In order to simulate wave wash on the shore, a volume of water is held back by a gate at the up-stream end of the tank, which gate is lifted suddenly, allowing a symmetrical wave to advance with uniform speed down the channel and impinge on the model of sandy foreshore and bulkhead.

Obviously, this tilting tank could be applied to a great variety of investigations, for example, for an extension to larger sizes of the experiments for determining the laws of flow in drainage pipes, made a few years ago by the U. S. Department of Agriculture. The pipe or conduit to be experimented upon could be blocked up by supports from the bottom of the tilting tank and the water could be run through it at any desired slope, from zero up to 3%, by quickly re-setting the screw supports, which could be done in a moment's time without disturbing the alignment. Such a series of experiments would give much valuable new data on the discharge of partly filled drains and sewers up to cross-sections 5 ft. or more in diameter.

Alongside this tilting-tank canal, I would propose building a parallel canal with concrete walls, say, 240 ft. in length, 15 ft. in width, and 15 to 25 ft. in depth, in which experiments could be made on the flow over models of dams and weirs of various forms, and in which other experiments could be made for determining new methods of precise measurement of velocity in disturbed currents. For certain tests, the current could be given a twist by inclined shear boards attached to the bottom or sides of the tank. In large, filled, circular conduits like turbine penstocks, there is often a surprising amount of twist in the current and sometimes it is present in deep rectangular canals.

There is abundant proof that some supposedly accurate measurements made by ordinary current meters in streams having deeply disturbed currents, have been seriously in error, and the use of such an experimental channel, it is believed, would be of great advantage to the Hydrographic Department of the U. S. Geological Survey, and to many other institutions, in educating its young men in the avoidance of errors in gauging, in addition to other uses in developing new and accurate methods.

In this second proposed experimental channel, provision should be made for weir experiments, with many varieties of shape and roughness of crest, and at greater depths than any heretofore made. A preliminary design for this second

experimental canal has been largely repeated from one prepared by myself ten years ago for a proposed large Hydraulic Laboratory for the Massachusetts Institute of Technology, which was crowded off the campus. This design proposes to circulate quantities of water as great as 600 cu. ft. per sec. by means of centrifugal pumps and to make possible the precise measurement of rates of discharge in a tank, with a limit of error within one-tenth of 1 per cent. It provides for experiments with 5 ft. in depth over a round-crested weir, 15 ft. in length, by utilizing a quick-swinging gate, somewhat after the idea used by Francis in his famous weir experiments, by which in a fraction of a second the weir flow can be diverted from one channel into another and the interval of tank-fill timed by an electric chronograph. This capacity of 600 sec-ft. could be used for experiments at greater depths over the weir by narrowing the sluiceway; also, various much needed data could be obtained on the coefficients of flow through sluices and over drowned weirs. Also, the new method of measuring depth on a round-crested weir, devised by Past-President Herschel, and partly tested under a grant from the Engineering Foundation,* could be thoroughly tested.

Many years ago, I became convinced that a standard round-crested weir could be developed that would be superior to the sharp-crested weir and that precision of measurement, particularly in inexperienced hands, can be greatly improved by thus avoiding the disturbances due to eddies and the irregularities in the contracted vein due to swirls and irregular motion in the channel of approach.

Certain preliminary experiments made by the late Mr. Mills, on a "Venturi flume", at Lowell, Mass., indicate that a very instructive series could be run on such a piece of apparatus in this new laboratory, which might develop considerable utility in the constructive arts.

The laws of the so-called "hydraulic jump", which has lately come into prominence as an absorber of energy and thus a safety device for protecting the undermining of dams, could be much better worked out. Through the mathematical investigations of Karl R. Kennison, Assoc. M. Am. Soc. C. E., reported to the Society,† we know that the time-honored formula is wrong and a series of experiments with varying forms of pit and with various forms of deflectors, at the foot of the fall, would aid greatly in showing how best to absorb energy in the friction of eddy currents with maximum structural economy. The absorption of energy at the "jump" comes largely from the friction losses in the violent churning and swirling of the waters in the small open chamber immediately down stream from the jet.

When a set of tests has once been worked out understandingly and carefully, the results are good for all time, within the limits tested, like the weir experiments made by Francis 60 years ago.

This proposed National laboratory would be essentially for experiments on a large scale, leaving to the small-scale college laboratories, about fifty in number, built in recent years, the researches within their capacity.

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* *Transactions*, Am. Soc. Mech. Engrs., May, 1920.

† *Transactions*. Am. Soc. C. E., Vol. LXXX (1916), p. 338

In order to give an example of the capacity of such a river laboratory,* let us consider the biggest problem possible, that of the Mississippi. Problems of German rivers have been studied in laboratory tanks of one-third the linear dimensions here proposed. And let us run an experiment from the point of view of flood relief rather than improvement of navigation.

In the beginning, however, let me say that if this experiment to be next described, was successful in the model, no means yet exist for carrying out such a plan of cut-offs and straightening on so grand a scale as the Mississippi would require. Groynes or retards cannot yet be built capable of holding a river bank 100 ft. high.

If the principle should prove good, there are many smaller rivers on which it could be tried. If the test failed, there might be achieved the valuable result of settling an argument that has been going on since the days of Galileo and of Ancient Rome.

Let there be modeled in the proposed tilting tank a winding stream with a soft or sandy bed, in form analogous to the Lower Mississippi, on a horizontal scale of 1 to 1 000 and a vertical scale of 1 to 100. This distortion of scale probably would not seriously disturb the results. It would be analogous to experimenting on the discharge over a section of dam crest only 3 ft. long, where the entire structure had a crest 30 ft. long. The test could be repeated on natural scale.

Our 200 ft. in length of experimental tank would then represent 200 000 ft., or about 40 miles, along the valley, and its 20 ft. of width would accommodate a bend with an ordinate of nearly 4 miles, while the broad part of the tank would take in a bend of nearly 10 miles ordinate.

Corresponding to the 100-ft. flood depth of the Mississippi, we would have a 1-ft. depth flowing in the tank, and instead of a main river channel, 3 000 or 4 000 ft. in width, the channel in the tank would be 3 or 4 ft. wide.

The slope of the tilting tank would be readily adjusted by the screws to give a velocity, somewhere near to that of the actual river; or such velocity that there would be an analogous slow process of erosion at bends and of deposition of sediment on the nearly opposite convex shoal, with an established regimen showing no important scour or fill along the straighter portions. Although swirls and vortices that exist in the large river, which have much to do with scour and transportation of sediment, might not be found fully represented, there doubtless would be some analogous action in the model stream.

The rise and fall of a flood could be represented by varying the discharge at the pump. Various percentages of sedimentary material could be put into circulation. The formation of bends and action of sand waves could be studied and a great variety of instructive observations made.

After having made various precise quantitative observations on the behavior of the model of the curving, sediment-carrying river in flood, let a section of channel comprising one or two bends be straightened by cutting off the bends and training it to the straightest practicable line, giving it the same average depth as before and holding it from spreading, or wandering horizontally.

* Outline designs for this laboratory are in course of preparation by the speaker, *pro bono*.

by means of dikes and projecting spur-dikes or groynes, which preferably should make the flood channel, say, 10 to 20% narrower than before, in order to encourage the scouring to greater depths.

At the down-stream end of this straightened section, let the water be restrained from changing its surface elevation, as by discharge into the Gulf, and at the up-stream end of the section straightened, restrain the bed from being scoured deeper by the swifter current by a "sill", analogous to that built at the head of the Atchafalaya outlet from the Mississippi to prevent its scouring deeper, or analogous to those built at the head of the "Passes".

During this rectifying of channel and enlarging of the cut-off trench some additional head for scour could be had temporarily, if needed, by the water backing up over the sill into the section next up stream, until erosion had lowered the bed.

In addition to increase of velocity and scouring force caused by increased slope, increased depth, of itself, tends to cause increase of velocity with increase of power to erode.

If under the original condition of the curving river, scour and sedimentation were in equilibrium and the regimen thus established over the erodible bottom, then under the new condition this increased current will dig the bed deeper than before, unless the river has a hard resistant bed.

As erosion of the bed progresses, the elevation of the water surface along the middle and up-stream part of the straight section will progressively become lower, because of the decreased slope needed to convey the water in the deepened channel. Rapids will be established over the sill placed previously at the head of the section, of increasing height and steepness, until equilibrium is established; and, finally, the current would probably settle down to nearly the same mean velocity all along the straightened river as that originally found in the crooked river, because with a given quality of bed material, the control and establishment of a permanent regimen depends on attaining the bottom velocity which is not great enough to dig and is too great to permit deposition. There is commonly some margin between these limits.

The feature of greatest interest and apprehension of trouble is the possibility that with the lessened obstruction from building up the crossing-bar in the curved channel, there would be substituted in the straight channel trouble from sand waves, moving irregularly here, there, or anywhere down stream, at the rate of only, perhaps, a mile per year. The experiment with the model might give some very useful suggestions. River observations have generally shown that the sand wave did not extend all the way across the channel, like the "crossing-bar".

The final longitudinal section of the straight river, after regimen is established, probably would differ substantially from that of the natural curving river which works itself into a succession of pools and shoals, deep pools in the bends, with their line of maximum depth following along near the concave shore, which pools are separated by bars of sediment, or coarser gravel, deposited at the place of reversal of curvature—the "cross-over", as the pilots call it—where the channel shifts to near the other bank. In the straight river, the trans-

verse section of the channel will be more nearly of uniform depth, and there will be no occasion for gravel bars to accumulate at the cross-over, for there will be no such strongly marked cross-over. Possibly much of this coarser material might find lodgment in the depths of the old pools. With the meander stopped, the supply of coarse material which now builds up the cross-over bars, would be lessened.

In the straight river, it is probable that the flood would dig the deposit of sediment more deeply and more uniformly all along, and that the increase of flood height caused by these submerged dams rolled up by the current during a flood, would be substantially reduced.

After one section of model river (or actual river) had been thus trained and its bed lowered, the section next up stream could be undertaken and the restraining sill between the two sections removed. At each successive stage up stream, there would be greater forces available for hastening the excavation, because the amount of fall over the successive sills would be cumulative; and the cumulative change of fall due to the lessened resistance to flow in a straight deep uniform channel and slope possibly would dig the bed enough deeper so that in the upper reaches its floods would no longer reach the top of the banks, and levees no longer be needed.

The reported experience of the past 20 or 30 years with the Atchafalaya channel, below the outlet sill from the Mississippi, indicates that the test would be successful.

The many disastrous effects recorded as resulting from cut-offs of bends in the Mississippi, in past years, seem to have been caused by lack of control. Things simply have run wild under the increased slope of the shortened piece of river. New bends have been eroded, banks undercut, vast areas of fertile land ruined, the products of scour deposited on bars down stream, and previously established regimen disturbed, it is said, for more than 50 miles, the disturbed length extending both above and below the cut-off, and the river has not appeared satisfied until the original excess of length has been restored. All this appears to be Nature's method of building deltas and fertile bottom-lands with material eroded from the hills, but after man arrives and cultivates these bottom-lands for a food supply, a change of program may be entirely reasonable. If a cut-off is to be beneficial, it must be under most carefully planned control, with sills, groynes, etc.

As stated previously, however, no practicable type of groyne has yet been developed, that could safely protect and restrain the 100-ft. depth of the Lower Mississippi. Apparently, there now is one that can cope with the Missouri and the Platte, and such laboratory test, therefore, would be applicable to those rivers and many others.

There are a hundred important problems that I have not space here to mention, the solution of which a laboratory on this large scale could hasten. Thoughtfully devised experiments in the laboratory compared with observations on the big river, would develop new lines of analogy, test theories, and surely bring benefits and economies far outweighing the cost of the laboratory, and give basis for deduction from observed facts, instead of mere personal opinions, speculation, and philosophical discussion.

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Paper No. 1508

Discussion*

CONSTRUCTION PROGRESS OF THE HETCH HETCHY WATER SUPPLY OF SAN FRANCISCO, CALIFORNIA

By J. D. Galloway, M. Am. Soc. C. E.

J. D. Galloway,† M. Am. Soc. C. E. (by letter).—The discussion by the author of this important work is very complete, and comment is limited to certain features of design, except that after one has seen the work in progress and after having read the description given by the engineer in charge, it is possible to express a considerable degree of appreciation of the character of the construction and the manner in which it is being conducted.

The author has mentioned the Board of Engineers employed to outline the general plan of the Moccasin Creek Power Plant, consisting of Professor William F. Durand, F. G. Baum, M. Am. Soc. C. E., and the writer. Some notes regarding this power plant and the reasons for its design and location may be of interest.

The general location of the aqueduct tunnel from Early Intake to the hills above Moccasin Creek was made by John R. Freeman, President, Am. Soc. C. E., in his admirable report on the entire project. As suggested therein, the plans called for the conduit tunnel to be driven near the hydraulic grade line to the point near the surface of the mountains. These series of tunnels, nearly 19 miles in length, and all under pressure, were to terminate in a surge chamber. From the surge chamber, the water was to pass to the power plant in a pressure tunnel, should the rock prove to be of sufficient strength to withstand the pressure of a static head of about 1 425 ft.

The study of this part of the project led the Board of Engineers to change the location of the tunnels, in order to use a regulating reservoir in place of a surge chamber and to substitute steel pressure pipes laid on the hillside for the pressure tunnel.

* This is a continuation of the discussion of the paper by M. M. O'Shaughnessy, M. Am. Soc. C. E., on p. 869.

† Cons. Engr., San Francisco, Calif.

The use of a regulating reservoir was advised as being in line with practice in California. An investigation of the characteristics of the average load in California showed that the daily load factor would be about 60 per cent. This daily load factor would not be available if the city carried out one possible disposal of its power and distributed it for lighting and street railway purposes in San Francisco. If this should be done, it was believed that the load factor would be not more than 44 per cent.

The subject of the necessary steam reserve plant at the distribution center was reviewed and the necessity of carrying some of the peak load by the steam plant was determined. If this was done, it was believed that a load factor of 65% could be maintained at the Moccasin Plant. Nothing that has happened since has changed this view, except that if the power was sold to the local power company, a higher system load factor would be available, and with the use of steam power to carry peak loads, the daily load factor on the Moccasin Plant might rise to about 85 per cent.

The prevailing conditions of load delivery being determined, it was evident that much potential energy would be wasted if dependence for load regulation was placed on the surge chamber alone. The function of the surge chamber is to care for momentary surges and changes of load rather than wide changes due to daily variation. It was recognized that the flow of water through the tunnels, as originally planned, could be varied to suit the load, but this would have required, for load regulation, a forebay of considerable capacity at the intake of the tunnels and a considerable variation in head on the wheels of the power plant, due to the varying friction loss in long tunnels. It would have been possible to construct the tunnel of a size sufficient to carry the maximum load without too great a friction loss, but the cost of such a tunnel was prohibitive.

For the reasons stated, but mainly to make available for sale all the potential energy in the water, it was decided to adopt a regulating reservoir if the physical conditions should permit it. A search was made of the locality and a fair site was found for such a reservoir. The location of the tunnel was changed, the westerly end being shifted south about a mile, where the Priest Regulating Reservoir is being constructed.

The calculations indicated that a reservoir with a capacity of 11 000 000 cu. ft., or 230 acre-ft., would be required to regulate the uniform flow of 620 sec-ft. in accordance with the load curve and load factor adopted. The reservoir is being constructed to have a capacity of 105 000 000 cu. ft., or 2 500 acre-ft. This will permit the regulation of the entire system of power plants proposed and possible, as those on the upper parts of the stream lack good sites for regulating reservoirs and will have to operate on practically unity load factor.

Consideration was given to the size of the tunnel. Taking account only of delivery of water to San Francisco, a smaller tunnel of a capacity of 200 000 000 gal. daily was advised. It was a wise decision to make the tunnel of full size for ultimate delivery, as it conserves water rights and permits of the full development of power at the plant. The resulting revenue will more than offset the increased cost.

The use of a pressure tunnel to the power plant was not considered advisable, because of the several failures of such devices, notably on the Los Angeles Aqueduct, where two tunnels failed under about two-thirds of the head proposed at Moccasin Plant, and also the failure, under less head, of a pressure tunnel on a plant of the Los Angeles Edison Company, on Kern River, many years ago. There has been some criticism of this change from the original plans, but the writer believes that the conclusions of the Consulting Board were correct.

The general layout of the power plant followed standard practice. The size of the generators was limited in one way to the greatest possible amount of power that could be developed by two single-nozzle, overhung water wheels attached to a generator with a horizontal shaft, under conditions of maximum efficiency.

Available Water.—There has been much controversy over the quantity of water available for the Hetch Hetchy Aqueduct. In granting the rights to the City of San Francisco to rights of way in the Yosemite National Forest, Congress insisted on recognition by the city of prior water rights in favor of the Turlock and Modesto Irrigation Districts, the canals of which take out below the city aqueduct. These imposed conditions are embodied in an Act of Congress called the Raker Bill.

It has been the writer's province, in connection with some other features of the project, to examine the effect of the conditions and the available water for San Francisco. It is unnecessary to set forth this detail study herein, but the conclusions are as follows:

(1).—There is ample water flowing in the Tuolumne River for all possible irrigation and for the delivery of 400 000 000 gal. per day to the city aqueducts.

(2).—On account of the nature of the Sierra Nevada streams, storage is imperative. To maintain a uniform delivery of 400 000 000 gal. per day, storage of 615 000 acre-ft. would have carried the city demand over the period of the three dryest years on record. With a reserve of 100 000 acre-ft., the total storage required would be 715 000 acre-ft.

(3).—The present Hetch Hetchy Reservoir of a capacity of 202 000 acre-ft., as described by the author, will maintain a supply of more than 200 000 000 gal. of water per day through a similar period of deficient stream flow. With the present rate of increase of population, it will be two or three generations before any additional storage will be required.

(4).—Ample storage sites are available, owned or controlled by the city, within the water-sheds to provide the required 715 000 acre-ft. when needed.

Policy.—Much criticism has been offered regarding the plan of this mountain supply for San Francisco and other cities on San Francisco Bay. Much of this criticism has been malicious and with evil intent. The comment offered by Allen Hazen,* M. Am. Soc. C. E., is not of this character, but there lingers in his statements some echoes of the battle that was fought before the rights to use the Tuolumne Reservoir sites was won.

* See p. 897.

It is axiomatic that the cheaper and near-by sources of water should be developed and used first, other things being equal. It is true, however, that had San Francisco waited until all the near-by sources of water supply had been developed and used, no water would have been left in the Sierra Nevada. At present, not a river is left in those mountains, within reasonable distance, that is not dedicated to irrigation in such a way that it would be impossible to obtain a supply therefrom.

Again, it may be said that all water will be costly in the San Francisco Bay region. The situation compels this result. The mountain supply, however, will be cheapened if the available power is developed and sold at a profit, as it can be. Critics of the project fail to take note of the credit from the power side. If developed and sold, at prevailing prices, sufficient revenue is possible from the Moccasin Power Plant alone to pay the interest on approximately $28 000 000 worth of 5% bonds, a sum that is close to the total cost of all works to that point. The physical situation is such that nearly the same return is available from another power plant above Early Intake, the costs of which would be only a fraction of the present installation.

Similar criticism has been directed against the policy of developing power along the line of the aqueduct. More than 200 000 h. p. will be available along the line of the completed works. It would have been an economic crime not to have developed and used such an amount of power as the needs require.

Much credit is due a number of engineers who have been connected with the project from its inception. It was conceived and placed in shape by C. E. Grunsky, M. Am. Soc. C. E., when City Engineer. Marsden Manson, M. Am. Soc. C. E., as City Engineer, kept the subject alive during the dark days when the city was in the control of interests adverse to the project. The able report of Mr. Freeman who placed the project on its present basis and gave the general outline that is being followed, was one of the determining factors that made the project a success. To the present City Engineer, Mr. O'Shaughnessy, is due the credit of conducting the work in the satisfactory manner described.

MEMOIRS OF DECEASED MEMBERS

JOHN FINDLEY WALLACE, Past-President, Am. Soc. C. E.*

DIED JULY 3D, 1921.

John Findley Wallace was born at Fall River, Mass., on September 10th, 1852. He was a son of David A. and Martha (Findley) Wallace.

In his early childhood, his father was sent to Monmouth, Ill., for the purpose of founding a college under the auspices of the United Presbyterian Church. Reared under the unusual social conditions formed by that combination of college, religious, and semi-pioneer life, the boy early manifested those manly, studious, and persevering qualities which, coupled with his high character and attractive personality, gave him the commanding position in the Engineering Profession which he so long enjoyed.

A student in Monmouth College until 1871, Mr. Wallace started his engineering career, in his nineteenth year, with the United States Corps of Engineers, and was engaged for five years on improvements of the Rock Island Rapids and the Upper Mississippi River and its tributaries.

For the following three years, he filled the positions of County Surveyor of Warren County, Illinois, and City Engineer of Monmouth, Ill., and, for five years thereafter, he was in charge of the location, construction, and operation of a railroad from Peoria to Keithsburg, Ill., which afterward was merged with the Iowa Central Railroad Company, and now forms part of the Minneapolis and St. Louis Railroad.

From 1883 to 1886, Mr. Wallace was associated with the Iowa Central Railroad Company as Chief Engineer of Construction, during which time he designed and constructed transfer facilities across the Mississippi River at Keithsburg, Ill., for that Company, and was Associate Engineer on the construction of a bridge, and its approaches, crossing the Mississippi River at that point.

From 1887 to 1889, Mr. Wallace was with the Atchison, Topeka and Santa Fé Railway Company, acting as Bridge Engineer on the extension of that Company's line between Chicago, Ill., and Kansas City, Mo., and having charge, as Resident Engineer, of the bridge built by the Company over the Missouri River at Sibley, Mo. During this period, he designed the foundations for the bridge over the Mississippi at Fort Madison, Iowa, and designed and had charge of the rectification works built in connection with the two bridges at Fort Madison and at Sibley.

From 1889 to 1891, Mr. Wallace was associated with the late E. L. Corthell, Past-President, Am. Soc. C. E., Consulting Engineer, in Chicago, and had charge of the construction of a joint entrance into the City of Chicago for the Atchison, Topeka and Santa Fé Railroad Company and a western connection for the Illinois Central Railroad Company. He acted as Associate with Mr.

* Memoir prepared by the following Committee: F. A. Molitor, E. J. Noonan, and H. R. Safford, Members, Am. Soc. C. E., and the late A. S. Baldwin, M. Am. Soc. C. E.

Corthell during this time on the foundation work of the Merchants' Bridge over the Mississippi River at St. Louis, Mo., and was likewise associated with him on various other works, including a proposed bridge over the Mississippi River in the vicinity of New Orleans, La.

In 1891, Mr. Wallace was appointed Chief Engineer of the Illinois Central Railroad Company, and immediately rendered distinguished service to that Company in the reconstruction of its terminals within the City of Chicago for the purpose of meeting traffic conditions arising through the location of the World's Fair adjacent to the tracks of the Company. In this connection, he was largely influential in the beginning of track elevation in Chicago, the Illinois Central, through his counsel and advice, being the first railroad company to enter into a contract for track elevation in that city.

During his service as Chief Engineer, the Illinois Central Railroad Company went through a period of great development in both construction and maintenance work; many branches and auxiliary lines were built, on many parts of the line, grades were reduced, alignment was improved, and second track was built. Many large yards were constructed, likewise large terminal facilities in New Orleans, La., St. Louis, Mo., and other places, and the general condition of the road was greatly improved.

In 1898, Mr. Wallace was transferred to the Executive Department, serving with the various titles of Assistant Vice-President, Assistant General Manager, and General Manager, until 1904, when he left the Illinois Central Railroad Company, having been appointed by the late President Roosevelt as Chief Engineer of the Isthmian Canal Commission. Later, he served as a member of that Commission, and was also a member of the Executive Committee and Vice-President and General Manager of the Panama Railroad and Steamship Company.

Mr. Stuyvesant Fish, at that time President of the Illinois Central Railroad Company, said of Mr. Wallace:

"There is no professional engineer I know who has in a higher degree than Mr. Wallace the special commercial and diplomatic tact needed on the new job. He has an excellent and rare capacity for dealing with men, above, beside, or below him, and he will be an honor to the Government and to himself."

There can be no doubt that to Mr. Wallace was largely due the credit for the organization originally built up for the construction of the Panama Canal, his knowledge of railroad transportation, a feature of prime importance, coupled with his wide and varied experience in engineering construction, rendered him peculiarly competent to organize and conduct a work of such great magnitude.

In 1905, Mr. Wallace relinquished his work at Panama and returned to the United States. On account of his reticence, his full reasons for resigning were never made public. It was known, however, that he was not in accord with the policy of the Administration in certain essential features connected with the plans and conduct of the work, notably the determination not to construct a sea-level canal.

In 1906, he acted as Confidential Adviser to the President of the Chicago and North Western Railway Company in the design of its new passenger terminals in Chicago, and very shortly thereafter he went to New York City to become President of Westinghouse, Church, Kerr and Company, at that time one of the largest engineering and construction organizations in the United States. During his association with that organization, it was engaged in the design and construction of engineering works of great magnitude in different parts of the United States and Canada, including the construction of terminal and dock facilities for the Canadian Pacific in various places, as well as work of the same character at different times for the Chicago, Burlington and Quincy, the Central Railroad of New Jersey, the Chicago and Alton, and the Baltimore and Ohio Railroad Companies. In addition to this railroad work, his Company constructed a number of large hotel, office, and factory buildings. During the ten years he directed the activities of this organization, it is estimated that its expenditures averaged more than $10 000 000 per year.

In 1913, Mr. Wallace was retained by the City of Chicago in an advisory capacity in connection with the granting of a franchise to the Union Station Company. This led to his permanent employment by the City as Chairman of the Chicago Railway Terminal Commission, a body of distinguished engineers, lawyers, and members of the City Council, which was organized to act in an advisory capacity to the City in all matters relating to railroad terminal facilities of magnitude within the city limits.

Mr. Wallace's unusual diplomatic ability, coupled with his experience in engineering and administrative capacities, well fitted him to harmonize the many conflicting interests involved in the great work of putting on a stable basis the relationship between the City of Chicago and the great railroad companies with which it was involved in transactions during this period. To his efforts, in no small measure, must be attributed the satisfactory solution of the Union Station problem and the arrangement made between the City, the South Park Commissioners, and the Illinois Central Railroad Company, culminating in what is known as the "Lake Front Ordinance", covering the extensive improvements required for the construction of a great system of parks on the Lake front by the South Park Commission; the relinquishment of its riparian rights; the electrification of its lines within the city limits; the building of a great passenger station by the Illinois Central Railroad Company; and the dedication for the benefit of the United States Government of a great harbor district.

In 1914, Mr. Wallace was head of the delegation made up of members of the Chicago Railway Terminal Commission, the Chairmen of various Aldermanic Committees, and other prominent citizens of Chicago, that visited Europe for the purpose of investigating terminal facilities of railroads and waterways and the construction of wharves, harbors, and docks. This investigation was broken up by the sudden declaration of war on the part of Germany.

Since 1915, Mr. Wallace had acted as Consulting Engineer, Director, and Member of the Executive Committee of the Taylor-Wharton Iron and Steel Company. For five years he was a member of the Committee of Harbors and Shipping of the Chamber of Commerce, State of New York. He had also been

Chairman of the Board of Directors of the Southern Oil and Transport Corporation and Chairman of the Board of Directors of the Century Oil Company.

During the World War, Mr. Wallace served as a member of the Transportation Committee of the National Petroleum Advisory Board.

At the time of his death on July 3d, 1921, he was in Washington attending the hearings of the Senate Committee on Interstate Commerce, in connection with the Board of Economics and Engineering which had been formed by the National Association of Railroad Security Holders, of which Board he was a member. The last words of his testimony before the Senate Committee were characteristic in voicing the earnest desire that he and the Board with which he was associated might be helpful in bringing together the various conflicting interests for the betterment of the transportation system.

He was married to Sadie Ulmer, of Monmouth, Ill., who, with a son, Harold Ulmer Wallace, and a daughter, Mrs. Pierre LeMay, survives him.

Mr. Wallace was a member of many social, professional, literary, and public clubs in the cities of New York, Chicago, and Washington, a list of which, together with a list of technical societies of which he was a member and a summary of the degrees that were conferred on him, is here given.

He was a member of the following clubs: (Chicago), Chicago Club, Union League, Engineers' Club, Association of Commerce, City Club, Press Club, and South Shore Country Club; (Washington), Metropolitan, and Cosmos; and (New York City), Union League, Sleepy Hollow Country Club, Engineers' Club, Engineers' Country Club, Bankers Club of America, Chamber of Commerce, Automobile Club of America, National Arts Club, and Railroad Club.

Mr. Wallace was also a Member, and Past-President, of the American Railway Engineering Association; Member, and Past-President, of the Western Society of Engineers, Chicago, Ill.; Member of the Institute of Consulting Engineers of America; and Member of the Institution of Civil Engineers of Great Britain.

Mr. Wallace held the degree of Civil Engineer from the University of Wooster, Ohio; the degree of Doctor of Literary Laws from Monmouth College, Monmouth, Ill.; and the degree of Doctor of Science from Armour Institute, Chicago, Ill.

His activities, as enumerated herein, are far from covering the wide range of service performed in a highly useful, energetic, and technical career of practically fifty years.

To close this brief memoir without some reference to those personal traits and qualities which endeared Mr. Wallace to his friends and distinguished him as a man, would be to give only an inadequate idea indeed of his character. An intimate associate for a number of years has given as his three outstanding characteristics:

First.—His interest in, and helpfulness to, young engineers.

Second.—His wonderful initiative and energy.

Third.—His unblemished character in every phase of his life.

In his treatment of young engineers, while consistently kind and fair, he was exacting, combining kindness with discipline. He demanded a full measure of service.

Mr. Wallace had a quality of cementing friendships to a rare degree. His characteristics in this connection were well illustrated in an address given by him to the graduating class of Monmouth College in 1916 in which he said: "My first advice to you would be to cultivate friends. Begin with your own family circle and let your friendships extend to include your neighbor, your city, your State, your country, and finally broaden it out to include and embrace humanity. Other things being equal, the measure of your success will depend upon your friendships."

The American Railway Engineering Association stands as one of the greatest monuments to his ability and skill as an organizer, as it was largely to the acumen and diplomacy which he evidenced in co-operating with other members of his Profession in the building up of this organization that its great success has been due, he having been one of the primary leaders in the movement and the first President of the Association. The result of the Association's work is outstanding among the professional societies of the United States. Its *Proceedings* constitute to-day the principal authority on which the present methods of railroad engineering are based. They have been commented on in international circles and, at one period during the World War, the United States Government ordered 2 500 copies of the "Manual" of the Association for distribution among the officers of the Engineering Corps.

During his residence in Chicago, Mr. Wallace exercised a potent influence on the affairs of the Western Society of Engineers, becoming its President, first in 1891 and again in 1896.

In 1900, the American Society of Civil Engineers was the guest of the Institution of Civil Engineers of Great Britain, holding its Annual Convention that year in the Society House of the Institution in London, at which time Mr. Wallace had the unique distinction of being the President of both the Society and the American Railway Engineering Association, giving the Annual Address of the former at this Convention in the Society House of the Institution.

Mr. Wallace contributed many papers of great value to the *Transactions* of the Society, as well as to the publications of the American Railway Engineering Association. He was one of the charter members of the American Institute of Consulting Engineers and was the first Chairman of its Committee on Professional Practice and Ethics, having been practically the author of the Code of Ethics which this Institute adopted.

Future historians of the Profession will class Mr. Wallace among the great engineers of the country. He was a keen student, an able administrator, an accomplished diplomat, and a gentleman of the highest type, and coupled with these qualities was a personality of great attractiveness.

His life will stand as an inspiration to the younger members of the Profession as illustrating the attainment possible through high character, a deep thoughtfulness in professional matters, and a genius for acquiring friends.

Mr. Wallace was elected a Member of the American Society of Civil Engineers on June 2d, 1886. He also served the Society as Vice-President in 1897 and 1898, as President in 1900, and as Treasurer in 1913 and 1914.

DAVID HERBERT ANDREWS, M. Am. Soc. C. E.*

DIED FEBRUARY 24TH, 1921.

David Herbert Andrews was born on September 17th, 1844, at Pepperell, Mass., where his father, the Rev. David Andrews, was minister of the Congregational Church.

In early life, Mr. Andrews, being inclined to mechanical pursuits, found employment in the machine shops of Fitchburg and Worcester, Mass. Believing that a college education would help him in his chosen line, he prepared himself by night study to enter the Chandler School of Science and the Arts connected with Dartmouth College, Hanover, N. H., and was graduated therefrom in 1869 with the degree of Bachelor of Science. In 1908, the Honorary Degree of Master of Arts was conferred on him by that institution. In 1896, he was made a member of the Board of Visitors of the Chandler School Foundation, which position he filled until his death.

After his graduation, Mr. Andrews taught Mechanical Drawing in a night school at Fall River, Mass., and, later, in 1871, entered the service of the National Bridge and Iron Works, at East Boston, Mass., as Engineer. In this capacity, he began a career notable for its part in developing the art of construction in iron and steel, at that time in its infancy. As one of the pioneers in this line, his natural talent for invention, his painstaking thoroughness, his industry, good judgment, and courage, helped him greatly in entering a field that was new to the Profession, using new materials, requiring new machines for fabrication, and new methods and appliances for erection. These pioneers were called on to design as well as to build, and they developed the theory as well as the practice of Structural Engineering in the United States.

About 1872, the National Bridge and Iron Works built the train-shed roof at its Boston Terminal for the Boston and Lowell Railroad Company. This roof consisted of braced arches of 116 ft. clear span and 63 ft. clear height at the crown, with framed purlin trusses and I-beam rafters. After forty-eight years of service, it was removed in 1920. The design was made by Mr. Andrews and, as the graphical method of determining stresses in braced arches had not been developed in this country at that time, he devised a scheme for making the necessary computations by studying the effect of a system of weights hung over sheaves on a catenary chain. The building of this roof caused considerable interest in engineering circles at the time, and the structure when built was the most noted of its kind in the country.

The National Bridge and Iron Works met with financial difficulties, and, in 1876, its affairs were wound up by Receivers. Mr. Andrews bought such tools and machinery from the Receivers as he deemed advisable, being obliged to borrow most of the capital therefor, and set up a plant, under the name of the Boston Bridge Works, at East Cambridge, Mass. As sole proprietor of the enterprise, he proved himself a business man of great ability,

* Memoir prepared by J. P. Snow, M. F. Brown, and John C. Moses, Members, Am. Soc. C. E.

soon achieving financial success as well as a wide reputation for the good workmanship of his structures, a large proportion of which were also designed by him and his engineers. This reputation and success was founded on his industry, strict attention to business, and to the confidence that he inspired in every one—employees, capitalists, supply firms, and customers—of his entire reliability and strict integrity.

A list of the work done during the first twenty years of his company's existence would comprise most of the railroad and highway bridges of New England and a considerable number in other parts of the United States. The first steel buildings in Boston would also be included. Mr. Andrews' talent for invention produced the first derrick car for bridge erection, in the early Eighties, and a new type of locomotive turntable which is still in wide use. In 1896, his plant was completely destroyed by fire, but he was not discouraged and immediately started plans to rebuild. After personally making a study of the best plants in the country, he designed his new factory and supervised its construction in every detail. In 1901, he incorporated The Boston Bridge Works, continuing as its President until his death and being succeeded in that office by his son, Mr. John G. Andrews.

Mr. Andrews' professional and business life covered a period of fifty years, during which time the use of iron and steel increased vastly. It required courage and industry, technical skill, and practical common sense to succeed in the early days, and this record is made as an example and encouragement for those who now take up the work that has been begun for them.

Mr. Andrews served as Chairman of the Chelsea School Board during his residence in that city. He moved to Newton Center, Mass., in 1890 and took an active part in the church and civic affairs of that community. He was elected a member of the Boston Society of Civil Engineers in 1881, and was a member of the Engineers' Club of Boston. He was also a member of the American Society of Mechanical Engineers, the Boston City Club, and of other civic and social organizations.

Mr. Andrews was married in 1872 to Miss Clara Gilbert, of Concord, N. H., who survives him, together with a daughter and three sons, two of the latter having been long associated with their father in his business.

Mr. Andrews was elected a Member of the American Society of Civil Engineers on September 2d, 1885.

CHARLES LeROY ANNAN, M. Am. Soc. C. E.*

DIED JANUARY 21ST, 1922.

Charles LeRoy Annan, the son of David E. and Lavina (Roberts) Annan, was born at Lawrence, Mass., on December 25th, 1853. He was of Scotch descent, although his ancestors had lived in New England for several generations.

After having been graduated from Worcester Technical Institute in 1876, Mr. Annan went West, and in November, 1878, began his railroad career as a Rodman on a location survey of the Atchison, Topeka and Santa Fé Railroad. During the next two years, he was engaged with this Company in various capacities as Rodman, Instrumentman, and Assistant Engineer, in the States of Colorado, Kansas, and New Mexico.

In June, 1880, he was appointed Office Engineer of the Sonora Railroad Company, at Hermosillo and Guaymas, Mexico, and after the completion of this road in December, 1882, took charge of the office work of the Chihuahua Division of the Mexican Central Railway Company.

In 1885, Mr. Annan accepted the position of Office Engineer of the Chicago, Burlington and Northern Railway Company, at St. Paul, Minn. In September, 1888, however, he severed his connection with that Company, to go with the Union Pacific Railroad Company at Omaha, Nebr., where he remained until May, 1889. He then returned to St. Paul, and was appointed as Office Engineer in the office of the City Engineer, the late L. W. Rundlett, M. Am. Soc. C. E. Mr. Annan continued in this position, with the exception of about four years, when he returned to railroad work, until March, 1912.

In April, 1912, he again returned to railroad work, this time as Chief Draftsman in the offices of the Great Northern Railway Company at St. Paul. He continued in the employ of that Company as Chief Draftsman and, afterward, as Valuation Engineer, until his death on January 21st, 1922.

He was married on April 30th, 1891, to Mrs. Margaret K. Baylor, who survives him.

Mr. Annan was of a quiet and reserved disposition, and only his closer associates knew of his extremely friendly and charming disposition. He was a conscientious and thorough workman in all matters which came under his charge. Having been a man who loved and cared for his home to the exclusion of public gatherings and public affairs, he found his greatest interest in his home life, although he was a member of the Masonic Order and had maintained his membership in the St. Paul Civil Engineers' Society for many years.

Mr. Annan was elected a Member of the American Society of Civil Engineers on July 4th, 1888.

* Memoir prepared by George L. Wilson, M. Am. Soc. C. E.

WILLIAM EDGAR BAKER, M. Am. Soc. C. E.*

DIED NOVEMBER 7TH, 1921.

William Edgar Baker was born on October 18th, 1856, at Springfield, Mass., in the home of his grandfather, General James Barnes. He was graduated from Lafayette College in 1877, with the degree of Civil Engineer.

Immediately following his graduation, Mr. Baker joined the Engineering Staff of the St. Paul and Pacific Railroad Company (now the Great Northern Railway Company) and served as Transitman and Engineer in charge of work until 1880.

He then entered the service of the Canadian Pacific Railroad Company as Transitman on surveys for that line through the Rocky Mountains. This work was conducted under such conditions of hardship that few of the members of the party survived, and Mr. Baker was soon in charge of the expedition. His diary written at this time has been given to the Engineers' Club of New York City. In 1883, he was appointed Resident Engineer in charge of bridges and buildings of the International and Great Northern Railway in Texas, which position he held until 1888.

In 1889, Mr. Baker began his career as an Electrical Engineer as Superintendent of Electric Car Service for the Thompson Houston Electric Company (now the General Electric Company), in Boston, Mass., where he had charge of the installation of the electric equipment of the West End Street Railway.

In 1892, he went to Chicago, Ill., where he designed and built the Columbian Intramural Railway at the World's Columbian Exposition, serving as General Manager of the railway until the close of the Exposition. This work was the first application of the third-rail system, and brought Mr. Baker into prominence as an Electrical Engineer both in the United States and in Europe.

The Metropolitan West Side Elevated Railroad, in Chicago, was under construction at that time, and although the Company had contracted for steam locomotives for use in its operation, Mr. Baker was asked to convert it into an electric road, using the third-rail system with which he had been so successful on the Intramural Railroad at the World's Fair.

In 1899, he came to New York City where he equipped the Manhattan Elevated Railroad with electricity. As General Manager of the Company, he operated the line until he resigned to open an office as a Consulting Engineer.

After engaging in private practice, Mr. Baker designed and supervised the construction of the electrical equipment of the Calumet and Hecla Copper Mines. He constructed at Chicago the first bascule bridge over the Chicago River, and was also consulted in connection with the construction of the London Underground Railways. After retiring from active business,

* Memoir compiled from information furnished by W. E. Baker and Company and on file at the Headquarters of the Society.

he spent much of his time at his country place at Chester, N. J., where he was interested in scientific agriculture.

Mr. Baker was married in 1884, to Miss Harriet Griffin, who, with five children and eight grandchildren, survives him.

He was a member of the Engineers' Club of New City and a Trustee of Lafayette College, in the welfare of which he was deeply interested.

Mr. Baker was elected a Member of the American Society of Civil Engineers on June 1st, 1898.

WARD BALDWIN, M. Am. Soc. C. E.*

DIED NOVEMBER 15TH, 1920.

Ward Baldwin, the son of Henry Ward Baldwin and Ann Esther (Van Ausdol) Baldwin, was born in Shrewsbury, Mass., on March 30th, 1856. In 1860, his parents moved to Cincinnati, Ohio, where his father died shortly afterward.

Early in life, Ward Baldwin showed an aptitude for mathematics and made rapid progress in his studies, having been graduated from the Hughes High School in June, 1874. From there he went to Worcester, Mass., where he was graduated from the Worcester High School, on July 15th, 1875. In the fall of that year, he entered the University of Cincinnati, from which he received the degree of Civil Engineer, and after taking a Post-Graduate Course, he received, on June 18th, 1880, the degree of Master of Science. In June, 1915, his Alma Mater conferred on him the degree of Doctor of Science. In 1883, he was appointed a Trustee of the University of Cincinnati, and served in that position until 1890, when he resigned to accept the chair of Civil Engineering.

During 1881, Mr. Baldwin taught Mathematics and Engineering at the University of Cincinnati, and in the same year he entered the service of the Cincinnati Southern Railroad Company, as Principal Assistant in its Engineering Department, where he remained until 1891.

In the fall of 1891, he returned to the University of Cincinnati, where he served as Registrar until 1896. From 1891 to 1900, he was also engaged as Professor of Engineering at the University, resigning to practice his profession as a Consulting Engineer.

As Consulting Engineer, Mr. Baldwin had charge of the design and construction of some of the largest industrial plants in Cincinnati and elsewhere in the United States. In this work, he showed such rare inventive genius, ripe knowledge, and fine executive ability, that his services were in constant demand.

On December 5th, 1902, he entered the employ of the Trustees of the Cincinnati Southern Railway Company as Consulting Engineer, and served them efficiently until his death. His work with this Company, constructing and supervising the terminals, viaducts, and bridges in Cincinnati, evidenced rare skill and unflinching fidelity.

He was still actively engaged in his work as a Consulting Engineer when, after only a brief illness, he died at his home in Cincinnati, Ohio, on November 15th, 1920.

As a writer for engineering publications, Mr. Baldwin's articles on bridge and concrete work placed him in the front rank of men of his Profession, who considered him competent authority on such subjects. His contributions to the *Transactions* of the Society included discussions on the following subjects: "Loadings for Railway Bridges", "Preservation of Railroad Ties", "Rail-

* Memoir prepared by Frank L. Raschig, M. Am. Soc. C. E.

way Bridge Designing" and "Train Loadings for Bridges". He also contributed a paper entitled "Stresses in Railway Bridges on Curves".*

Resourceful, patriotic, and untiring, Mr. Baldwin was ready at all times to serve his city or its University, and gave them his services without stint. It is related that his work in rebuilding the large auditorium for the Sængerfest in 1900, which had collapsed, proved him to be a prodigy. For that service, he was presented with a Gold Medal by Local Societies.

Under his enthusiastic leadership, there was secured from the City of Cincinnati the grant of the site in Burnet Woods Park, where the buildings of the University are now located, and he did much personally toward supervising the erection of these buildings.

Mr. Baldwin followed his profession not merely for the income it brought him, but as a pursuit worthy of his entire mind and heart. He was a fine engineer, holding his profession in the highest regard and insisting on the same consideration for it from all with whom he had to do. He was thoroughly conscientious, incorruptible, energetic, and a hard worker. Although he took an active interest in public improvements, he was most unassuming, and never indulged in display or rhetoric. His life was an example and an inspiration.

Mr. Baldwin was elected a Junior of the American Society of Civil Engineers, on March 2d, 1881, and a Member on October 2d, 1889. He was also a member of, and the first President of, the Cincinnati Section of the Society, having held that position at the time of his death.

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* *Transactions*, Am. Soc. C. E., Vol. XXV (1891), p. 459.

ELBRIDGE HARLOW BECKLER, M. Am. Soc. C. E.*

DIED AUGUST 26TH, 1908.

Elbridge Harlow Beckler was born at Boston, Mass., on October 16th, 1854. He was graduated from the Maine State College of Agriculture and Mechanic Arts at Orono, Me., in August, 1876, with the degree of Civil Engineer. Previous to entering college, he had completed a course at Maine Wesleyan Seminary, Kents Hill, Me., where special attention to mathematics and some experience in surveying, enabled him to complete the four-year engineering course in two years.

In 1877, having spent a few months in teaching, Mr. Beckler went to Minnesota to engage in railway construction, but as business conditions at that time were unfavorable, he was compelled to take other work—teaching, farming, surveying, and map-making—during the next two years.

From May 17th to December 24th, 1879, he served as Transitman on location, and Assistant Engineer on construction, on the St. Paul, Minneapolis, and Manitoba Railway, between Fergus Falls and Barnesville, Minn. From March 15th to July 1st, 1880, he was employed as Transitman on location, on the Yellowstone Division, for the Northern Pacific Railroad Company, and from July 1st, 1880, to January 5th, 1881, he was Assistant Engineer in charge of a party on location for the same Company.

Mr. Beckler was engaged as Assistant Engineer on Location on the Rocky Mountain Division of the Northern Pacific Railroad, from March 15th, 1881, to February, 1882, and from the latter date until January 24th, 1884, he was Resident Engineer on Construction. This work covered 41 miles and included the Bozeman Tunnel, 3 610 ft. long.

From April 1st to September 1st, 1884, Mr. Beckler served as Division Engineer on the Canadian Pacific Railway, along Kicking Horse River, on the western slope of the Rocky Mountains. This work included a few miles of grading and the construction of several wooden bridges. From September, 1884, until January, 1886, he was Resident Engineer on Construction of the St. Louis River Bridge, for the Northern Pacific Railroad Company, between Duluth and West Superior, Minn.

On January 6th, 1886, Mr. Beckler was appointed Resident Engineer on Location and Construction for the Montana Central Railway, which position he held until December, 1886, when he was made Assistant Chief Engineer, serving as such until November, 1887. This was the beginning of the Great Northern Extension to the Pacific Coast. In 1889, he became Chief Engineer, in charge of the entire work from Central Montana to Puget Sound.

In addition to a record for efficiency and economy in the accomplishment of this work, which has seldom been equalled, he secured a route through the main range of the Rocky Mountains, with lower gradients than any other transcontinental line. The location for most of the distance was through an unexplored and mountainous country, making the problem exceedingly

* Memoir compiled from information on file at the Headquarters of the Society.

difficult, but the entire distance of 818 miles was completed in a little more than two years from the time construction work was inaugurated.

In 1893, Mr. Beckler moved to Chicago, Ill., where he was engaged as Consulting Engineer on many projects, among which was the construction of a narrow-gauge railway in Arizona for the United Verde Copper Company.

From 1895 until 1902, he was in the service of Winston Brothers, Contractors, and on March 1st, 1902, at the time of the organization of the Company as a Corporation, he became a Stockholder and a Director.

He died suddenly of cerebral hemorrhage on August 26th, 1908, at the West Portal Camp of the St. Paul Pass Tunnel, in Shoshone County, Idaho, where he was engaged on work in connection with the Coast Extension of the Chicago, Milwaukee, and St. Paul Railway.

Mr. Beckler was married in 1880 to Miss Mera Rogers, of Richmond, Me., who, with two daughters, survives him.

He was a man of great energy and high ideals, sympathetic, and considerate. He was an Honorary Member of the Montana Society of Civil Engineers and a member of the Western Society of Engineers.

Mr. Beckler was elected a Member of the American Society of Civil Engineers on April 6th, 1892.

JAMES RICHARD BELL, M. Am. Soc. C. E.*

DIED JULY 14TH, 1913.

James Richard Bell was born at Wick, Scotland, on August 21st, 1841, and was educated at Aberdeen University.

He served for a time with an architect in Liverpool, England, and afterward with John Gardiner, a Railway Engineer, in London. After engagements as Assistant and as Resident Engineer on railways in England, Mr. Bell went to Egypt where he was interested in many railway projects.

In 1868, he joined the Indian Public Works Service, in which Service he remained for many years. He was one of the pioneers in railway construction in India, bridging rivers in the north under most adverse circumstances, on which work he was compelled to use untrained and unskilled native labor. He was employed at first in the Madras Presidency, but, later, was transferred to the State Railways Department of India, in which service he remained until his retirement in 1896.

Mr. Bell did excellent work and gained a high reputation in bridge construction. Among the more important of his bridges may be mentioned the Empress Bridge over the Sutlej, the Muttra Bridge across the Jumna, and the Ferozepur and the Sher Shar Bridges over the Chenab. He also devised methods for training rivers by means of bunds.

He made many difficult surveys and constructed many miles of railway on the Northwestern, East Coast, Indus Valley, and other Indian Railway Systems, one of his most important services to India having been the construction of the line from Ruk to Bolan, which gave military access to Kabul. This work was accomplished in an exceedingly short time.

Mr. Bell served the Indian Government for several years as Consulting Engineer for State Railways and Harbors. In 1896, his health broken by overwork and the hot enervating climate, he retired from active service and returned to London, England, where, he lived until 1901. He then took up his residence at Ightham, Kent, England, where he died on July 14th, 1913.

He was a man of exceptional energy and ability, stern, but kindly, devoted to duty, and greatly respected. He was the original of the character "Findlayson" in Rudyard Kipling's story, "The Bridge Builders", contained in his "Day's Work".

Mr. Bell was elected a Member of the American Society of Civil Engineers on September 2d, 1896. He was also a Member of the Institution of Civil Engineers of Great Britain.

* Memoir compiled from information furnished by E. Brook-Fox, Assoc. M. Am. Soc. C. E., and on file at the Headquarters of the Society.

FRANK EDWARD BISSELL, M. Am. Soc. C. E.*

DIED FEBRUARY 25TH, 1922.

Frank Edward Bissell, the son of Thelus Martin Bissell and Ellen Roxana (Stedman) Bissell, was born at Rootstown, Ohio, on August 21st, 1855.

He grew up at South Bend, Ind., his boyhood home, and was prepared for college in its High School. In 1874, he entered Cornell University, taking the course in Civil Engineering, and was graduated in 1878, with the degree of B. C. E. He returned to Cornell for one year of additional study and, in 1879, took the advanced degree of C. E. As an undergraduate, he acquired high standing in his classes and was named by the late Dean E. A. Fuertes, M. Am. Soc. C. E., as Computer for the survey work in his Senior year—the second highest honor. He was an active member of the Zeta Psi Fraternity.

Immediately on his final graduation in June, 1879, Mr. Bissell was employed in the U. S. Engineer Corps as Rodman on construction work on the Missouri River at Atchison, Kans. He was made, successively, Instrumentman and Acting Assistant, and finished this work in December, 1879.

In 1880, he joined the forces of the late Jay Gould, on what afterward became the latter's Southwest System of railroads, with headquarters at Fort Worth, Tex. For three years, Mr. Bissell served as Draftsman, Assistant Engineer, and Chief Draftsman. In these positions, he showed himself to be systematic, tireless, of keen perception and an invaluable office man, and these traits he never lost. He afterward became Resident Engineer on maintenance of the Missouri, Kansas, and Texas Division of the Gould lines, and during the next three years was engaged on construction work. From 1885 to 1888, he was employed on the location and construction of the greater part of the Fort Worth and Denver City Railway, about 350 miles, under the late Gen. Grenville M. Dodge, Hon. M. Am. Soc. C. E., who was then President of that railway. It was pioneer work, and Mr. Bissell won the highest confidence of Gen. Dodge. He had good business qualities; he seldom forgot; he never hesitated; he always worked; he was square; and his honesty was never once in question.

From 1888 to 1891, his work was in the Operating Department of the road he had built—soon afterward controlled by the Union Pacific Company—as Superintendent and Chief Engineer of Construction.

During the following six years, Mr. Bissell was engaged as Engineer for Industries, Superintendent, and Valuation Expert for various companies in the West and Southwest until 1897, at which time he was appointed Engineer for the Receiver of the Wheeling and Lake Erie Railroad, at Cleveland, Ohio, where he remained until 1900, having been engaged on maintenance work and on valuation work of various lines under contemplation of purchase by that Company.

In 1900, at the instance of the late Gen. Dodge, he resigned to accept the position of Chief Engineer of the Arkansas Northern Railway—a line of

* Memoir prepared by Willard Beahan, M. Am. Soc. C. E.

considerable magnitude which never passed the preliminary stage. Mr. Bissell then took part, as Field Inspector, on the first State valuation of railroads in the United States, namely, that of the State of Michigan, under Dean M. E. Cooley, M. Am. Soc. C. E., of the University of Michigan.

In 1901, he became First Assistant Engineer of the Lake Shore and Michigan Southern Railway, and during the ensuing six years, he did important work as a Staff Officer and also completed the location and built the "Low Grade", a line from Ashtabula to Youngstown, Ohio, the Franklin and Clearfield, and commenced the Cleveland Short Line, around the City of Cleveland, Ohio.

At the inception of the valuation of railroads of the United States by the Federal Government, Mr. Bissell was made Senior Assistant Civil Engineer of the Central District, with an office at Chattanooga, Tenn., in charge of all the field parties in civil engineering work. This work required about six years, and on its completion, he returned to Cleveland, where his family had continued to reside. He then became Chief Engineer of a Contracting Company, which position he retained until seized with his fatal illness. This seemingly unimportant attack was the first illness he could recollect, and his death on February 25th, 1922, was unexpected.

He was married on April 21st, 1886, to Ida Mayer Smith, at Detroit, Mich., who, with their son, Howard Mayer Bissell, survives him.

Mr. Bissell was a member of the American Railway Engineering Association, the American Association of Engineers, and the Cleveland Engineering Society, having been a member of the Board of Direction of the latter at the time of his death. He was a Thirty-second Degree, Scottish Rite Mason, Knight Templar, and a Noble of the Mystic Shrine.

Mr. Bissell was an engineer of unusual natural ability. He had had opportunities of learning railroading from its masters in certain pioneer branches, and he had profited by these experiences. He was a worker, a thinker, and earnest in all he did. Honesty was to him inborn; he hated sham, and was no diplomat. He made some enemies, which fact had hindered him at times; but he made his mark on American railroads, and that can never be effaced.

Mr. Bissell was elected a Junior of the American Society of Civil Engineers on April 2d, 1884, and a Member on September 2d, 1891.

GEORGE PIERREPONT BLAND, M. Am. Soc. C. E.*

DIED APRIL 18TH, 1921.

George Pierrepont Bland was born in Roxborough, Philadelphia, Pa., on December 30th, 1851. He received his early education at the public schools of that place, after which he entered Lehigh University and was graduated from that institution, with the Class of 1872, with the degree of C. E.

Immediately after his graduation from Lehigh, Mr. Bland entered the employ of the Pennsylvania Railroad Company, under the late Joseph M. Wilson, M. Am. Soc. C. E., then Engineer of Bridges and Buildings, and was engaged in office work incidental to the design, erection, etc., of the bridges used on its main line and branches.

In 1876, Mr. Bland accepted an appointment as Engineer for Cofrode and Saylor (The Philadelphia Bridge Works), a bridge-building firm, at Philadelphia, Pa. Subsequently, he entered the general contracting business under the firm name of Gibson and Bland which, later, became Bailey, Milliken and Bland.

In 1895, Mr. Bland organized and established the Keystone Structural Company, the fabricating shops of which are located at Royersford, Pa. As President of this Company, he had offices in Philadelphia where he attended to the making of contracts and the designing and detailing of structural steelwork for buildings, bridges, etc., continuing as the active head of the business until his sudden death from angina pectoris on April 18th, 1921.

Mr. Bland was married on February 3d, 1875, to Miss Alice A. McCalla, who, with one daughter and two sons, survives him.

While at Lehigh he was noted as one of its most brilliant students and was generally liked and esteemed. As an alumnus, he was always interested in University affairs and responded liberally to calls for support and aid.

Mr. Bland's tastes were markedly scientific, and during his life he was rarely without some mathematical work which he used mainly as a recreation from professional and business cares. Although he was a good executive and managed his business with care and skill, he was primarily a student, which trait had developed more and more during the later years of his life.

In disposition, Mr. Bland was retiring, and although he was sufficiently aggressive to conduct his business successfully, he was by no means what is called a good "mixer"; after his early friends and comrades had passed away, he did not easily form other friendships, and his new connections were usually of a business nature.

Mr. Bland was elected a Junior of the American Society of Civil Engineers on April 7th, 1875, and a Member on May 4th, 1881.

* Memoir prepared from information furnished by Dr. Henry S. Drinker, President Emeritus, Lehigh University, South Bethlehem, Pa., and on file at the Headquarters of the Society.

ALFRED PANCOAST BOLLER, M. Am. Soc. C. E.*

DIED DECEMBER 9TH, 1912.

Alfred Pancoast Boller was born in Philadelphia, Pa., on February 23d, 1840. His early education was received at the Episcopal Academy of his native city, followed by his graduation, with the degree of A. B., from the University of Pennsylvania in 1858. He then entered Rensselaer Polytechnic Institute from which he was graduated with the degree of C. E., in the Class of 1861. It is interesting to note that among his classmates at Rensselaer were Estevan A. Fuertes, M. Am. Soc. C. E., William L. Haskins, Robert Neilson, M. Am. Soc. C. E., T. Guilford Smith, M. Am. Soc. C. E., and William N. Simmington, all of whom are now dead.

Soon after his graduation, Mr. Boller began his engineering career as a Rodman on the Nisquehoning Railroad, advancing to the positions of Instrumentman and Topographer. In the latter capacity, he made an elaborate survey and map of the middle and southern anthracite coalfields for the Lehigh Coal and Navigation Company, and, in 1862, was employed on the repairs to the Company's Canal, following the great flood that damaged and destroyed parts of it.

In 1863, he entered the service of the Philadelphia and Erie Railroad Company, in the Department of Bridges and Buildings, and from this period he turned his attention chiefly to structural engineering. During this engagement, he was detailed to inspect and report on the harbor facilities of various cities on the Great Lakes, with reference to proposed harbor improvements at Erie, Pa.

In the summer of 1865, Mr. Boller was in charge of the construction of a suspension highway bridge at Williamsport, Pa. In the spring of 1866, he was appointed Engineer of Bridges on the Atlantic and Great Western Railroad and, during this engagement, planned the Cattaraugus Viaduct and an international bridge over the Niagara River at Black Rock, N. Y. After the failure of this railroad enterprise, he became, in the fall of 1866, Chief Engineer of the Hudson River Railroad, but resigned after about six months to associate himself with Samuel Millikin as Agents of the Phœnix Iron Company. During the four years of this partnership, Mr. Boller was connected, either as Engineer or Contractor, with a number of enterprises, among which were the Bridgeport Bridge, the construction of Piers 38 and 39 on the North River, and the design of the St. John's Park Station, of the New York Central and Hudson River Railroad, in New York City.

In the summer of 1871, he accepted the position of Vice-President and Engineer of the Phillipsburg Manufacturing Company, engaged in the design and construction of bridges and other structural iron work, serving in that capacity until its failure in the panic of 1873. During his connection with this Company, he designed and supervised the construction of a number of railroad and highway bridges and other structures, among the former being the Park Avenue Bridge over the Morris Canal at Newark, N. J., which had some

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

novel features, described in a paper presented before the Society.* Following the collapse of the Company, he served as Chief Engineer of the Manhattan Elevated Railroad, the Yonkers Rapid Transit Commission, and the West Side and Yonkers Railroad.

In 1874, he opened an independent office in New York City and soon acquired a large and important professional practice, which he continued until his death, at his home in East Orange, N. J., on December 9th, 1912.

Among the more notable enterprises on which Mr. Boller was engaged may be mentioned: the Staten Island Rapid Transit Railroad; the New York, Providence and Boston Railroad; the Thames River Bridge, involving difficult and novel foundation work and approaches; Consulting Engineer to the Department of Parks and to the Department of Public Works, New York City; to the Rapid Transit Commission (New York City) of 1884; to the contractors for the foundation of the Statue of Liberty, New York Harbor; the Albany and Green Bush Bridge; the West Side and Yonkers Railroad, including the Eighth Avenue Bridge over the Harlem River, New York City; the bridges for the New York Central and Northern Railroad over the Bronx River and at Croton Lake†; the Congress Street Bridge at Troy, N. Y.; the Eastern Avenue Draw Bridge at Boston, Mass.; the Harlem River Bridge at 155th Street and Seventh Avenue (more commonly known as the Central or Macomb's Dam Bridge), New York City; the Arthur Kill Bridge, New York City; and the bridge over the St. Louis River, at Duluth, Minn. Mr. Boller was also a member of contracting firms for the construction of the Bergen County Branch of the Erie Railroad, and the foundations for the great gas tanks of the Bay State Gas Company, at Boston, Mass.

For a number of years, he was the Consulting Engineer for the various projects and improvements of the Lake Superior Company, at Sault Ste. Marie, in Canada and Michigan. He was retained, in a number of cases, as Consulting Engineer by the United States Government and by the State of New York. He also acted as Consulting Engineer on a number of very deep and difficult building foundations in New York City.

In 1898, Mr. Boller formed a partnership with the late Henry M. Hodge, M. Am. Soc. C. E., under the firm name of Boller and Hodge and, in 1912, Howard C. Baird, M. Am. Soc. C. E., who had been connected with the work of the office for some time, was admitted to the firm, which then became Boller, Hodge, and Baird, Mr. Boller continuing as the senior member.

Under the first named firm, the bridge over the Arkansas River at Little Rock, Ark., was designed and built, as well as the important bridge over the Monongahela River and that over the Ohio River at Mingo Junction for the Wabash Railroad, at Pittsburgh, Pa.; and the bridges for the Algoma Central Railroad in Canada, the Municipal Bridge over the Mississippi River at St. Louis, Mo., the State Bridge over the Connecticut River at Saybrook, Conn., and a number of bridges for the City of New York, were designed. In 1907, the firm was appointed as Consulting Engineers to the Joint Commission of

* *Transactions*, Am. Soc. C. E., Vol. II (1873), p. 379.
† *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 150.

the States of New York and New Jersey to report on plans for interstate bridges over the Hudson River at New York City. The firm also acted as Consulting Engineers on the steel framework for the Singer Building and the Metropolitan Life Building, in New York City. During this period, Mr. Boller served as Consulting Engineer on the new masonry arch bridge at Hartford, Conn., which was completed in 1907, and is commonly considered to be the finest example in America of the stone arch bridge.

Under the firm of Boller, Hodge, and Baird, construction work on a number of the previously named structures, including the Municipal Bridge at St. Louis, was prosecuted, and the bridge over the Connecticut River at East Haddam, Conn., and the concrete bridge over the Hillsborough River at Tampa, Fla., were designed and built. Mr. Boller and his firm acted as Consulting Engineers for the National Railways of Mexico and designed bridges for railroads in South and Central America, Cuba, the Philippines, and Haiti, as well as many minor railroad and highway bridges in the United States.

Mr. Boller was recognized as standing in the front rank of structural engineers and as an expert in bridge engineering. Familiar with the technical side of the art and science, he was especially noted for his practical good sense and sound judgment. Not a few of his bridges were characterized by their originality and boldness of design. The draw-span of the Thames River Bridge, 503 ft. in length, and weighing 1 200 tons, was the longest attempted until that time. Another remarkable structure, the great viaduct (or Central Bridge) over the Harlem River at 155th Street, New York City, 4 500 ft. long, costing over $2 000 000, and having a draw-span weighing 2 400 tons, was stated at the time to be the heaviest movable mass in the world. The channel span of the Municipal Bridge over the Mississippi River at St. Louis, was at the time it was designed the longest fixed truss span in the world, 670 ft. between centers of piers. To the great advances made in bridge design and construction during his professional life, Mr. Boller contributed his full share. He was the author of a small book, published in 1876, entitled "Practical Treatise on the Construction of Iron Highway Bridges, for the Use of Town Committees." This book was chiefly intended for the information and assistance of municipal officials in that line of their duties.

Mr. Boller was especially well qualified by natural endowments, culture, and training for his vocation. His mind was keen, logical, and persistent. His judgments and opinions were the result of careful and deliberate consideration, rather than of hasty impulse or impression. He had an intuitive sense of, and fondness for, the artistic, and one of his leisure diversions was the sketching in water-colors of landscapes, at which he was notably successful for an amateur. His appreciation of architectural symmetry had a marked influence on his bridge designs, his constant effort being to combine technical principles and practical utility with symmetrical and pleasing outlines. Notable examples of his success in these efforts are the 155th Street Bridge in New York City and the Hartford Bridge.

He was married on April 28th, 1864, to Katharine, daughter of William Henry Newbold, of Philadelphia, Pa. Mrs. Boller and three sons and two daughters survived him.

Mr. Boller was an exemplary citizen. For fifty years, his home was in a suburb of New York City, which grew, in that time, from a mere village to an important suburban city, and in the inception and development of its public works and civic enterprises he took a keen interest, and his counsel and aid were always sought and freely given. As the first President of the first Shade Tree Commission in the State, he organized it on an efficient and successful basis. Few civic organizations or enterprises were considered complete without his name and influence. For a long period, he was a member and influential Vestryman of Grace Protestant Episcopal Church, one of the leading churches of the vicinity.

In his relations with his fellow engineers and associates, Mr. Boller was courteous, considerate, and helpful, particularly to the younger members of the Profession, many of whom remember with gratitude the aid and encouragement extended to them.

In business matters, he was diligent and efficient, and especially careful to be just and fair to every one.

No term seems so fittingly to describe his personal character and social qualities as the word lovable. His fondness for, and his devotion to, his home and his family were notable traits in his character.

He was active in the organization of the American Institute of Consulting Engineers, serving two terms as its first and second President. He was also a member of the Institution of Civil Engineers, and belonged to the Century Club and to other clubs in New York City and vicinity.

Mr. Boller was elected a Member of the American Society of Civil Engineers on December 4th, 1867, soon after its resuscitation and reorganization, following the Civil War. He served as its Secretary, without compensation, in 1870-71, as a Director in 1872, and as a Vice-President in 1911-12, his death occurring during his term of office. He was a member of the Special Committee on the Means of Averting Bridge Accidents (1873) and of the Special Committee on Steel Columns and Struts (1909). He took an active interest in the affairs of the Society, contributed several papers to its *Transactions*, and frequently took part in the discussion of professional subjects.

Mr. Boller's long and busy life exemplified the best traits of American manhood and citizenship. As an engineer, not many have contributed more to the usefulness, dignity, and honor of the Profession.

WILLIAM HENRY BOOTH, M. Am. Soc. C. E.*

DIED NOVEMBER 12TH, 1921.

William Henry Booth was born at Rochdale, Lancashire, England, on September 30th, 1854. He was educated at Rochdale Grammar School and Owen's College, Manchester (now the Victoria University), and was apprenticed to Messrs. John Petrie and Company, Ironfounders, at Rochdale.

In 1877, Mr. Booth went to New Zealand and thence to Sydney, New South Wales, where he was employed in the Locomotive Works of the State Railways until June, 1880. He then returned to England, and for the next three years was engaged with the Manchester Steam Users Association, a pioneer body, in the United Kingdom, in the systematic inspection and insurance of steam boilers and engines. He was afterward a partner in a contracting firm which built the sewage outfall tunnel at Rochdale, and a sea outfall sluice at Gibralter Point in Lincolnshire.

In 1887, Mr. Booth went to New York City, but he returned to England in 1889. In 1907, he again visited New York in connection with the utilization of blast-furnace gas.

After assisting with the preliminary engineering work on a subway in the west of London, which scheme failed to obtain Parliamentary sanction, he accepted an appointment with Messrs. Legrand and Sutcliffe, the well known Well and Water Engineers. This engagement gave scope to, and enabled him to enlarge, his knowledge of the geology of England, on which subject he had remarkably detailed information. He particularly studied water-bearing strata, and could give an opinion of the probability of the success of well-boring, and the depth at which water would be found, in any locality, with great accuracy. About 1899, and for about three years following, he was a member of the Engineering Staff of the British Electric Traction Company, mainly engaged on power-house design, but owing to the wide range of his professional knowledge, his work with that Company included many other subjects.

In the early years of the Twentieth Century, Mr. Booth took the leading part in an exploration of the hinterland of Venezuela, *via* the River Orinoco, the territory covered being prospected for mining and general commercial purposes. If this expedition has not been successful, it is because of difficulties in the way of commerce, which may be generally described as political. When the time comes, his report of this journey should be of great value.

Mr. Booth was a man of wide knowledge. He could not be fairly described as a specialist in any one branch of Engineering, because such a description is altogether too narrow. He had the knowledge and vision which enabled him to see how any engineering question was related to others, and hence to exercise a fully informed judgment which took account of all the relevant circumstances. He was, therefore, an all-round expert, from whom one could rely on obtaining a plan for anything he studied, which took account of all the conditions, giving proper weight to each.

* Memoir prepared by Henry M. Sayers, Esq.

Mr. Booth's connection with engineering journalism was a long one, his first article appearing in the *English Mechanic* in 1876. He was on the staff of the *Mechanical World* from 1881 to 1887, for four years as Editor. For many years, he contributed to the *Railroad Gazette* and to the *American Machinist*. He was also on the editorial staff of the *Electrical Review* (London) for a number of years.

He devoted a great deal of study to the economical utilization of fuel, smoke prevention, the construction of furnaces, especially for steam boilers, and chimneys. He was much interested in fuel economy, and advocated methods for attaining that end, much in advance of contemporary practice, which methods are now recognized as standard.

His last professional work was done in charge of the water supply to the great motor-car repair depot at Slough, where he had the satisfaction of sinking a deep well which struck water-bearing strata within a few inches of the depth which he had estimated.

Mr. Booth was a Fellow of the Geological Society of London. Geology, especially in connection with water supply, was the subject in which he perhaps took the greatest interest, and was most widely known as an expert.

He published several books, among which are the following: "Liquid Fuel and Its Combustion", "Liquid Fuel Apparatus", "Smoke Prevention and Fuel Economy", "Superheat and Superheating", and "Steam Pipes". He was a contributor to Kempe's "Engineers' Year Book" from its earliest issue, and the 1921 edition contains articles by him on Steam Engines and Boilers; Properties of Gases; Pumps; Cotton Mills; Water Softening; and Well Sinking. He also contributed numerous papers to and lectures before English engineering societies.

Mr. Booth was married in 1877, and is survived by his widow, two sons, and one daughter. He was a truly lovable man, the most loyal of friends, and the most chivalrous of opponents.

Mr. Booth was elected a Member of the American Society of Civil Engineers on July 4th, 1888.

JAMES SIMPSON BROWNE, M. Am. Soc. C. E.*

DIED OCTOBER 22D, 1921.

James Simpson Browne, the son of the Rev. Faneuil Browne, a Presbyterian clergyman, was born on November 14th, 1862, in Indiana, Pa. He received his early education in the schools of Indiana County and was tutored by his father in the higher branches.

Mr. Browne entered on his career as a Civil Engineer with one of the pioneer parties of the Union Pacific Railway, and was employed on the location and construction of its line through the Dakotas, Wyoming, and Idaho. He was afterward engaged in the development of the elevated railways in Brooklyn, N. Y., and in the building of the Washington Arch over the Harlem River, New York City, at that time the largest steel bridge span in the United States. Later, he joined the forces of the late George B. Francis, M. Am. Soc. C. E., in laying out the West Shore Railroad along the Hudson River.

In 1889, Mr. Francis was engaged by the late E. P. Dawley, M. Am. Soc. C. E., as Resident Engineer of the Providence Terminal Company, which was organized to design, construct, and operate a Union Passenger Station to serve the railroads then entering the City of Providence, R. I., and now forming parts of the New York, New Haven, and Hartford Railroad System, and Mr. Francis brought Mr. Browne to Providence to assist him in that work.

When the station was completed, Mr. Browne was retained by Mr. Dawley, largely in connection with the abolition of grade crossings on the New York, New Haven and Hartford Railroad in Rhode Island and Massachusetts, and as Assistant in charge of maintenance work.

In 1903, a Division Office of the System was established in Providence, with Mr. Browne as Division Engineer. He served in that position until 1914, when he was made Assistant to W. J. Backes, M. Am. Soc. C. E., Engineer of Maintenance of Way, and moved to the headquarters of the Department at New Haven, Conn., where he served until his death.

Although Mr. Browne had had a wide and varied experience in railroad location and construction, having begun his career in what was then the Far West, under the strenuous conditions surrounding pioneer railroad location work, and having had a part in the construction of important engineering structures, he was essentially a student, and preferred research work in the quiet of his office or study, rather than the turmoil of the construction camp.

As Assistant to the Engineer of Maintenance of Way, one of his first and most important duties was to prepare a manual for the guidance of the employees, and the present "Book of Rules" of the Maintenance of Way Department of the New York, New Haven and Hartford Railroad Company, is a monument to his devotion and loyalty. Devotion and loyalty are the very foundation of an engineer's training and, as a rule, are taken as a matter of

* Memoir prepared by H. L. Ripley and W. S. Nichols, Members, Am. Soc. C. E.

course and hardly mentioned, but they were so conspicuous an attribute of Mr. Browne's character that they are the first qualities that come to the minds of his associates in the Profession, who thus pay spontaneous tribute to the man.

Mr. Browne is survived by his widow, Mrs. Flora D. Browne, two brothers in Kansas City, and two brothers and a sister in Pennsylvania. He was buried in the family plot at Huntingdon, Pa.

He was a member of the American Railway Engineering Association and the American Bridge and Building Association.

Mr. Browne was elected an Associate Member of the American Society of Civil Engineers on October 4th, 1893, and a Member on July 1st, 1909.

HARRY DEAN BUSH, M. Am. Soc. C. E.*

DIED MARCH 15TH, 1922.

Harry Dean Bush, the son of Austin Ballou and Susan E. (Millard) Bush, was born on April 2d, 1857, in Springfield, Mass., and was descended on both sides from old Massachusetts families of Colonial distinction. He received his preparatory education in the elementary and High Schools of his native city. Later, he attended the Worcester Polytechnic Institute, from which he was graduated with honor, in 1879, ranking second in his class, with the degree of Bachelor of Science.

Mr. Bush entered on his career as a civil engineer in the old R. F. Hawkins Bridge Works in Springfield, Mass., and became, at the end of two years, Bridge Engineer for the Northern Pacific Railroad Company, with headquarters at Portland, Ore. After four years at Portland, he was, for one year, Assistant Engineer in the office of the late George S. Morison, Past-President, Am. Soc. C. E., in New York City, and, for three years, Superintendent of the Bridge Shops of the Dominion Bridge Company, Limited, at Montreal, Que., Canada.

After terminating his connection with the Dominion Bridge Company, Mr. Bush spent a year of travel and study in Europe, and on his return was again associated, for two years, with Mr. Morison, in New York City. He then went again to Portland, Ore., where, for three years, he was Engineer and Superintendent for the contractors who constructed the Bull Run Pipe Line, 24 miles long, for the Portland City Water-Works System, and the steel gates for the Cascade Locks, Columbia River, the largest ever built previous to those for the Panama Canal.

Mr. Bush then became Contractor for the pipe line for the Water-Works System of New Bedford, Mass., and, in 1899, after an absence of ten years, returned to the Dominion Bridge Company as Engineer in charge of the erection of bridges. In this capacity, he had charge of the erection of the Royal Alexandra Bridge, across the Ottawa River, at Ottawa, Ont., Canada. He presented a paper before the Canadian Society of Civil Engineers on the construction of this bridge, for which he was awarded the Czowski Silver Medal, an annual prize given the writer of the paper or article of the highest merit on some phase of engineering.

In 1903, Mr. Bush went to Baltimore, Md., and became Vice-President and Manager of the Baltimore Bridge Company, then recently organized. It was through his untiring energy and devotion that this Company soon acquired a well deserved reputation of the highest standing, furnishing steel bridges and buildings for the Baltimore and Ohio and other railroads, the notable steel arch bridge over the spillway of the Croton Dam for the New York City Aqueduct Commission, and much work for export, including all the bridges on the Guatemala Railway System, bridges in Costa Rica, sugar-mills in Cuba, and several important contracts for the Isthmian Canal Commission. It was largely as a result of his work that the Baltimore Bridge

* Memoir prepared by Joseph E. Lewis, Esq., Baltimore, Md.

Company was acquired by the Carnegie Steel Company about 1912, and, as a consequence of this new connection, the scope of the plant was greatly enlarged under his continued management.

In the spring of 1911, a new industry was brought to Baltimore from Newark, N. J. This new company was known as the Tube Bending and Polishing Machine Company and was backed financially by Mr. Miner C. Keith of New York City, the President and owner of the Baltimore Bridge Company. Mr. Bush became President of this new company as well as of the Compressed Copper Company, a subsidiary organization. The Tube Bending and Polishing Machine Company was re-organized later into the Baltimore Tube Company, Incorporated.

Mr. Bush was married (first) to Emma F. Witherbee, of Gardner, Mass., who was well known for her musical ability. Mrs. Bush died in 1907, and in 1909, Mr. Bush was married to Frances (Dent) Davis, the daughter of Thomas Dent, of St. Mary's County, Maryland, who survives him.

Mr. Bush was an engineer of high standing and a business man of no mean ability. Physically, he was compactly and strongly built, of vigorous and alert bearing, kindly countenance, and affable manners. Of genial disposition, he was popular socially and had a large circle of friends. He was pre-eminently a friend of men, especially of young men, and was actively interested in the Young Men's Christian Association. He was a man of sterling qualities, whose word was as good as his bond. It has been said of him that he never missed an engagement and was never late at an appointment. A Democrat in politics, he took a lively interest in all matters concerning the public welfare.

His special tastes were for Science and Mathematics. Mr. Bush was always a great reader and, in the course of his life, had collected a large library of essays, romance, poetry, and the drama. His home reading was chiefly outside the field of his work. He was also a lover of music and of culture.

Liberal in his views and true to the traditions of his family, his church affiliation was always with the Universalists or Unitarians. During the last few years of Walt Whitman's life, Mr. Bush was one of his supporters and attended several of the birthday dinners given to the "Good Gray Poet", including the one made famous by the debate between Whitman and Ingersoll on "Immortality."

He was a member of the Engineering Institute of Canada, the Engineers' and Whitehall Clubs of New York City, and the Baltimore Athletic Club.

Mr. Bush was elected a Member of the American Society of Civil Engineers on May 2d, 1888.

FREDERICK WILLIAM CAPPELEN, M. Am. Soc. C. E.*

DIED OCTOBER 16TH, 1921.

Frederick William Cappelen was born at Drammen, Norway, on October 21st, 1857. He received his high school education at Frederickstad, Norway, and was graduated from the Technical School of Oerebro, Sweden. He afterward attended the Polytechnicum at Dresden, Germany, from which he was graduated as a Civil Engineer with the highest honors ever won at that time by a foreign student at the school.

In 1880, Mr. Cappelen, came to the United States and was employed in the Engineering Offices of the Northern Pacific Railroad Company in New York City and at Brainerd, Minn. From 1881 to 1884, he served as Assistant Engineer on location and construction of the Missoula Division of the Northern Pacific Railroad, in Montana, and from 1884 to 1886, he was engaged, as Assistant Engineer, on bridge work with the St. Paul and Northern Pacific Railway Company.

He then entered the employ of the City of Minneapolis, Minn., as Bridge Engineer, remaining in that position until 1892. During this time, among other work, he designed and built three highway bridges across the Mississippi River.

On January 2d, 1893, Mr. Cappelen was elected City Engineer of Minneapolis and held that office until 1898. In 1913, he again became City Engineer, remaining in office until his death. During this latter term, he built the St. Anthony Bridge, which crosses the Mississippi at St. Anthony Falls. This bridge is a fine example of reinforced concrete construction, 2 223 ft. long and 80 ft. wide, and is conceded to be a noble and enduring structure.

The Franklin Avenue Bridge across the Mississippi, another reinforced concrete highway bridge, 1 030 ft. long with five spans, the central one of which is 400 ft.—one of the longest concrete spans in the world—was designed by Mr. Cappelen, and was in the process of erection under his direction at the time of his death.

In 1895, he designed the Minneapolis Reservoir System which was the first step toward the purification of the water supply of that city. His interest in and designs for the improvement of the city water-works, caused him often to be referred to as the "Father" of the present system.

From 1898 to 1913, Mr. Cappelen maintained an office as a Consulting Engineer on bridge and municipal work. During this time, he had a large part in the bridging and lowering of the steam railroad tracks through Minneapolis, and also acted as Consulting Engineer for the Great Northern Railway Company.

In 1904, he served on a commission with Mr. Andrew Rinker, then City Engineer, and Allen Hazen, M. Am. Soc. C. E., of New York City, for the investigation of a pure water supply for Minneapolis, and their report recom-

* Memoir prepared by George L. Wilson, M. Am. Soc. C. E.

mended the purification of the present supply from the Mississippi River. In 1913, the operation of the filtration plant was begun under his direction, and in 1921, its capacity had been increased to 90 000 000 gal. per day. During the thirteen years in which Mr. Cappelen served Minneapolis as City Engineer, the water-works system was rapidly extended and improved, keeping pace with the growth of the city.

From 1907 to 1911, he was also connected with the Decaries Incinerator Company, working out, during this time, many improvements in its garbage reduction process.

Extensive studies and investigations were made by Mr. Cappelen on the subject of grade separation at street and railroad crossings and a part of this work has already been brought to a successful completion; general plans for much more were completed by him shortly before his death.

His standing and reputation as a Sanitary Engineer was recognized by the Governor of Minnesota, who, in 1918, appointed him as one of the first two engineer members of the State Board of Health, and in January, 1921, he was again appointed to this position.

He was married in 1883 to Miss Felicitas Wessel, of Dresden, Germany, who, with two sons, Arthur S., of New York City, and Felix G., of Akron, Ohio, survives him.

Mr. Cappelen was a member and Trustee of the American Water Works Association, a member of the American Society of Municipal Improvements and of the Minneapolis Engineers' Club. He was also a member of the Odin Club, of the Masonic Order, the Elks, and many civic organizations.

His place in the estimation of the public was well stated in a leading city paper on the day following his death:

"For a quarter of a century this man had given his best in public service to Minneapolis. He used his engineering genius, not merely to facilitate communication, but to preserve public health, to promote safety and to enhance the attractions of the city. His devotion to the duties of his office and to the upbuilding of Minneapolis was not without its personal sacrifices. His passing is a loss that will not easily be retrieved."

Mr. Cappelen was elected a Member of the American Society of Civil Engineers on April 3d, 1895.

ELIOT CHANNING CLARKE, M. Am. Soc. C. E.*

DIED MAY 4TH, 1921.

Eliot Channing Clarke, the son of the late Rev. James Freeman and Anna (Huidekoper) Clarke, was born on May 6th, 1845, at Boston, Mass. The first ten years of his life were spent in Boston and in Meadville, Pa., the home of his mother's family. In 1855, his family settled in Jamaica Plain, Mass., where he attended the public schools and was fitted for college at the Eliot High School. He was graduated from Harvard College in 1867, serving as Chief Marshal of his class.

Mr. Clarke then took special studies at the Massachusetts Institute of Technology, and in February, 1868, began civil engineering work on a bridge, then building, over the Mississippi River at Quincy, Ill. Among other works with which he was connected later were the bridge over the Mississippi River at Hannibal, Mo., the Decatur and East St. Louis Railroad, the Phœnix Iron Works, the Detroit River Tunnel, the Chicago Water-Works tunnels, and the Chicago sewerage system.

In July, 1876, he was appointed Engineer in Charge of the survey for a Main Drainage System for Boston, Mass. The project having been adopted, he began construction in 1877, and the System was completed and put in operation early in 1884. Later in that year, Mr. Clarke became Chief Engineer of the Massachusetts Drainage Commission which was appointed to design methods of preventing the pollution of the water in the Charles, Mystic, and Blackstone Rivers Basins. In 1886, he was appointed a member of a commission to devise a plan for preventing floods in the valley of Stony Brook.

In December, 1886, and until 1904, he managed mill properties at Lowell, Mass., having served at different times as Treasurer of the Lowell Bleachery and the Boott Cotton Mills. In 1904, he retired from active business and, thereafter, until his death on May 4th, 1921, he attended only to his private affairs, with an office and residence in Boston and a summer place in Tamworth, N. H.

Mr. Clarke was fond of country life, of riding, and of fine horses of which he owned many. He had served as a Governor of the Riding Club of Boston.

He was a Fellow of the American Academy of Arts and Sciences and its Treasurer for eleven years. He was also a member of the Massachusetts Natural History Society, the Massachusetts Horticultural Society, the Colonial Society, and the Corporation of the Massachusetts Institute of Technology. He had also served as a Trustee of the Massachusetts School for the Feeble-Minded, Trustee and Vice-President of the Provident Institution for Savings, and as a Director of the State Street Trust, and other companies.

In 1885, Mr. Clarke was awarded the Norman Medal by the Society for his paper "Record of Tests of Cement Made for Boston Main Drainage Works, 1878-1884"†. He had also published an account of the Main Drainage Works

* Memoir compiled by Arthur T. Safford, M. Am. Soc. C. E.

† *Transactions*, Am. Soc. C. E., Vol. XIV (1885), p. 141.

of Boston, and had written many other technical papers and discussions. Of his little treatise on "Astronomy", 12 000 copies were sold.

In 1878, Mr. Clarke was married to Alice de Vermandois Schier, by whom he had five children, three of whom survive him, namely, Susan Lowell Clarke, James Freeman Clarke, and Mrs. Charles Eliot Ware, Jr.

Mr. Clarke was elected a Member of the American Society of Civil Engineers on September 4th, 1878, and served as a Director in 1889. He had also served as a member of the Special Committee of the Society on a Uniform System for Tests of Cement.

THOMAS CURTIS CLARKE, M. Am. Soc. C. E.*

DIED MAY 25TH, 1921.

By the death on May 25th, 1921, of Thomas Curtis Clarke, the Engineer Officers' Reserve Corps lost an officer of the highest standard; the country lost a citizen who had given completely of his services as an engineer in peace and war and who was an advocate and real worker in the cause of National defense; and the Society lost a member who, in his specialty of metallurgical research, can ill be spared to the Profession.

Thomas Curtis Clarke was born in Philadelphia, Pa., on December 11th, 1873, his father having been the late Thomas Curtis Clarke, Past-President, Am. Soc. C. E.

The son of one of the most distinguished engineers of his time, Mr. Clarke, like his brothers, was educated in the Engineering Profession, having taken a special course in Metallurgy at the Massachusetts Institute of Technology. After leaving the Institute, he became for a time a Chemist at the furnaces of the Union and South Works of the Illinois Steel Company, but returned to the Institute to complete some special studies. For a few years after leaving the Massachusetts Institute of Technology, in 1893, Mr. Clarke served as an Inspector and Assistant Engineer on the construction of the Third Avenue and Willis Avenue Bridges across the Harlem River in the City of New York. Between 1897 and 1902, he was engaged in business in which, however, his engineering education was used. In 1902 and 1903, he was Treasurer of the Imboden Coke and Embree Iron Company, and had engineering charge of the construction of the furnaces of the Embree Iron Company.

In 1904, Mr. Clarke was made Treasurer and Assistant General Manager of the Astoria Steel Company. Later, he was asked to take charge of the construction and installation of the Illinois Steel Company's coal washers at Danville, Ill., a position in which his unbounded energy and experience was given full play. During 1905, he was engaged as Assistant Superintendent of the Lackawanna Iron and Steel Company, at Lebanon, Pa., in charge of five blast furnaces and by-product coke-oven plants. It was during this period, that he became interested in the specialty of coal and coke by-products, which specialty led him into much foreign travel, in investigations of by-product coal and coke plants in Germany, France, and other Continental countries.

In 1912, Mr. Clarke became interested and took charge of the development of the Niagara Company, comprising a by-product coke plant, at Buffalo, N. Y. This plant was being built under German plans and patents, but was stopped, however, at the outbreak of the World War in August, 1914, as it was being constructed largely with German capital.

Mr. Clarke had become interested in military affairs as early as 1895, when he joined the National Guard of the State of New York, and was an active student of military affairs and operations.

In 1915 and 1916, foreseeing with a military eye, and aided by his experience in Germany and his acquaintance with the German people, that it was

* Memoir prepared by F. A. Molitor, M. Am. Soc. C. E.

inevitable that the United States would become an active participant in the World War, he devoted his energies and time to the preparedness movement. In 1916, he assisted in the organization and training of engineer battalions in New York City, and was commissioned a Captain in the Engineer Officers' Reserve Corps when it was formed through a provision of the National Defense Act of 1916.

On the entry of the United States into the war, Capt. Clarke, with many other engineers of New York City, was ordered to the First Officers' Training Camp at Fort Oglethorpe, Ga., in May, 1917. On his graduation from this camp in August of that year, he was named as Adjutant of the 104th Engineers, a regiment organized from this training camp. Later, he was promoted to be Lieutenant-Colonel and assigned to the 110th Engineers which was the Divisional Engineer Regiment of the 35th Division. This regiment went overseas with the Division in the spring of 1918. When Col. Cheney was promoted to the rank of Brigadier General, Lt.-Col. Clarke succeeded to the command of the regiment with the rank of Colonel from August 7th, 1918. Col. Clarke commanded this regiment in the Meuse-Argonne Offensive. The regiment performed the usual work of an engineer regiment attached to an active combatant division and, in addition, it was called as infantry into the lines where it took over a sector of the front line in the active offensive then in progress.

Col. Clarke occupied as his post of command a German "pill-box", the German artillery directing their fire thereon, making it necessary for him and his immediate staff to change their post of command during the height of a severe artillery fire. The regiment acquitted itself with great credit with a loss of more than three hundred men in this emergency, two officers and seven enlisted men having received the Distinguished Service Cross for their work. The regiment was cited for its service, and Maj.-Gen. W. C. Langfitt, U. S. A., M. Am. Soc. C. E., Chief Engineer, American Expeditionary Forces, commended it, as follows:

"Before issuance of definite orders for your regiment to return to the States, it is my desire that the command be advised that they have met the conditions imposed by the conflict just concluded in a most satisfactory manner.

"The construction of field fortifications and bridges across the Somme, the front-line construction, tunneling, and road work in the Vosges were notably well done. The excellent record made in front-line service is a matter of pride to the Chief Engineer as it should be to each soldier in your regiment.

"I desire that you and your command know that the services rendered were highly satisfactory and deserve commendation."

Early in Col. Clarke's service with the American Expeditionary Force, and before the Argonne Offensive, he was decorated with the Croix de Guerre by the French Government as a result of his voluntary participation in a raid which broke through the German lines.

After the Armistice, Col. Clarke was relieved from duty with the 110th Engineers, and was appointed Acting Deputy Director of the Army Trans-

port Service, at Tours, France, serving as such until March 31st, 1919, when at his own request he was ordered home and mustered out of the service. His interest, however, in future preparedness, and in the Engineer Officers' Reserve Corps, prompted him to take a commission as Colonel in that Corps, the rank which he held at the time of his lamentable death.

After being honorably discharged from the service, Col. Clarke became Vice-President of the International Coal Products Company, in direct and immediate charge of the engineering work. He held this position until his illness required him to relinquish it. An operation for an intestinal disorder resulted in pneumonia which caused his sudden death, his fatal illness having been due undoubtedly by the fact that he had been gassed repeatedly while in service and when pneumonia developed, medical skill could not save him.

Col. Clarke's genial personality, wit, and unfailing patience and good nature endeared him to his hosts of friends both in and out of the Profession, and they mourn his loss.

He was married on July 21st, 1897, to Elizabeth I. Knox who, with a daughter, survives him. He is also survived by two brothers, Mr. E. A. S. Clarke, for many years President of the Lackawanna Steel Company and now President of the Consolidated Steel Corporation, and Lt.-Col. Herman Clarke, of London, England.

Col. Clarke was a member of the Metropolitan Club and the Army and Navy Club of New York City, and the Rumson Country Club. He was also a member of the American Institute of Consulting Engineers, Society of Chemical Industry, Society of American Military Engineers, and the Military Order of the World War.

Col. Clarke was elected a Member of the American Society of Civil Engineers on May 4th, 1909.

FRANK HUDSON CLEMENT, M. Am. Soc. C. E.*

DIED FEBRUARY 18TH, 1922.

Frank Hudson Clement was born in Philadelphia, Pa., on June 6th, 1853, of Quaker stock. He was educated in the schools of that city, having been graduated from the Lincoln Grammar School.

Mr. Clement started on his long and successful career as an employee with Naylor and Company of Philadelphia, manufacturers of iron and steel, from 1869 to 1874.

In 1874, he studied the practical end of Civil Engineering and Surveying with Hudson Shedaker, and in October of that year, he entered the employ of the Pennsylvania Railroad Company, with which he remained until January, 1878.

During 1876 and 1877, under Assistant Engineer W. L. Ziegler, the late Stacy B. Opdyke, Jr., M. Am. Soc. C. E., and others, Mr. Clement was engaged on the construction of the stockyards at Philadelphia and of the West Philadelphia Depot, and, in 1877, he was in charge of the construction of the Pittsburgh Depot.

Early in 1878, he joined the ill-fated expedition of the Madeira and Mamoré Railway Company in Brazil. This expedition was the result of a contract made by Messrs. Philip and Thomas Collins, for the construction of a railroad around the falls and rapids of the Madeira and Mamoré Rivers. The party was composed of men, all of whom, in later years, became prominent in various walks of life. The expedition failed, owing to lack of sanitation and inability to transport supplies.

Early in 1879, Mr. Clement entered the service of the Northern Pacific Railway Company as Assistant Engineer and Engineer to the Vice-President, and, in 1880, was Topographical Engineer, under Col. J. T. Dodge, on surveys for the Missouri and Yellowstone Divisions.

In 1881, he was engaged as Engineer in charge of surveys and locations on the Mexican National Railway, and Division Engineer on the completion of the Toluca Division. He returned to the United States in 1882, as Division Engineer of the South Penn Railroad, spending four years on surveys, location, and construction.

Later, Mr. Clement returned to the East and, having entered the construction business on his own account, built a part of the New York Aqueduct. On the completion of this work, he went to Cuba with the Cerebro Company to take entire charge of its mines, after which, on returning to the United States, he contracted to build 35 miles of railroad in Maryland.

In 1890, he formed a partnership with Mr. John C. Rogers, of New York City, to build the first Niagara Falls Tunnel for the Niagara Falls Power Company.

Mr. Clement was also associated as a partner, with Mr. Charles F. King, under the firm name of Clement, King, and Company, and after Mr. King's death, he continued the business under the name of F. H. Clement and

* Memoir prepared by L. V. Morris, M. Am. Soc. C. E.

Company. Until 1906, Mr. Clement and his associates undertook and completed work costing many millions of dollars. It is worthy of note at this time that the contractors for the Madeira and Mamoré Railroad evidently recognized his ability, as they were associated with him in all the larger projects.

Mr. Clement and his associates specialized in heavy railroad construction, so that to-day the Mt. Airy and Sand Patch Tunnels on the Baltimore and Ohio Railroad, the Galitzin and Baltimore Tunnels on the Pennsylvania Railroad, the New York Central (Beech Creek), into Clearfield, Pa., and the Petersburg Cut-off of the Pennsylvania Railroad, are monuments to his ability.

From 1910 until a year previous to his death, Mr. Clement was engaged in industrial work, principally for the Bethlehem Steel Corporation at Bethlehem, Pa., Steelton, Pa., and Sparrows Point, Md., contributing to the great expansion of that Corporation's plant. During the World War, he built for the same firm a shell-loading plant at Mays Landing, N. J.

Mr. Clement's long and prominent career was the result of elementary training, the overcoming of seemingly insurmountable difficulties, which, in after years, made the detail of his organization and work a simple matter. One of his chief characteristics was his ability to make and retain friends. For thirty-one consecutive years, at his Port-Au-Peck Clambake, were gathered as his guests, intimate friends, the highest exponents of many professions and activities.

In 1885, Mr. Clement was married to Anna Louise Dietz, who survives him.

The character of the man is indicated by his social, fraternal, military, civic, and technical affiliations. His clubs were the Castine Golf Club, of Maine; Royal Societies Club, of London, England; Huddersfield Fish and Game Club, of Canada; Engineers Club of New York; Engineers Club of Philadelphia, Pa.; Art Club, of Philadelphia; New York Athletic Club; National Republican Club, of New York City; Bethlehem Club, of Bethlehem, Pa.; Garden City Club, of Long Island; and the American Cornish Club, of Utica, N. Y.

He was a member of Everett Lodge F. and A. M. No. 524, of Everett, Pa.; Bedford Royal Arch Chapter No. 225, of Bedford, Pa.; and the Masonic Veterans, of New York City. He also belonged to the following military organizations: New Jersey State Rifle Association, of Sea Girt, N. J.; Old Guard, City of New York; First Regiment (Veteran Corps) N. G. P., and Old Guard, Co. K, and Co. A, First Regiment, N. G. P., of Philadelphia, Pa.

He was also a member of the following civic organizations: Madeira-Mamoré Association, of Williamsport, Pa.; Society of Friendly Sons of St. Patrick; and the Contractors Association, of Philadelphia; as well as the following engineering societies: American Institute of Mining Engineers, Franklin Institute, Verein Deutscher Eisenhüttenleute, and the American Concrete Institute.

Mr. Clement was elected a Member of the American Society of Civil Engineers on November 1st, 1882.

FRANCIS COLLINGWOOD, M. Am. Soc. C. E.*

DIED AUGUST 18TH, 1911.

Francis Collingwood, the son of Francis and Elizabeth (Kline) Collingwood, was born in Elmira, N. Y., on January 10th, 1834. His father, a watchmaker and jeweler, was a native of Manton, Rutlandshire, England, and came to America about 1814. His mother was from Elmira, N. Y., her ancestors having come to the United States from Germany, and having been among the early settlers in this country. Her father served in the Revolutionary War.

After attending the Academy in Elmira, Mr. Collingwood entered Rensselaer Polytechnic Institute, at Troy, N. Y., and was graduated therefrom at the head of his class in 1855, with the degree of C. E. He then went to Wisconsin, where he was occupied in railroad engineering work, and, later, became City Engineer of Elmira, N. Y.

For about ten years ending in 1869, he was engaged in the sale of scientific instruments, but left that business to accept a position as Assistant Engineer on the East River Suspension Bridge, New York City, where he remained until July, 1883. His work in connection with this structure had to do more especially with the sinking of the caissons and the building of the towers, the anchorages, and the New York City approach, which was designed entirely by him. He was one of the first engineers to sink very large caissons, and those used in laying the foundations of the bridge towers were the largest ever constructed at that time. When this vast work was completed, Mr. Collingwood was occupied in making extensive repairs to the Allegheny Suspension Bridge, at Pittsburgh, Pa., which had suffered from rusting of the cable wires at the anchorages.

Beginning in 1885, he became a regular contributor to the *Sanitary Engineer,* and, at the same time, he opened an office in New York City as Consulting and Expert Engineer. In this capacity, Mr. Collingwood gave testimony in many cases of back-water, infringements of water rights, masonry failures, arches, etc., advised in cases of flood protection, was consulted in the case of the Johnstown Flood, was an Expert Examiner respecting frauds in the New Croton Aqueduct, and was employed as Consulting Engineer in important sewer work.

He wrote many papers concerning bridge work, particularly caisson, tower, and anchorage work, which have been published by the Society.† He also prepared a paper on the repairs to the Allegheny Suspension Bridge for the Institution of Civil Engineers for which he was awarded the Telford Medal and the Telford Premium.‡

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* Memoir compiled from information furnished by W. P. Mason, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

† "A Few Facts About the Caissons of the East River Bridge", *Transactions,* Am. Soc. C. E., Vol. I (1872), p. 353; "Foundations for the Brooklyn Anchorage of the East River Bridge", *Transactions,* Am. Soc. C. E., Vol. III (1874), p. 142; Vol. IV (1878), p. 205; "Further Notes on the Caissons of the East River Bridge", *Transactions,* Am. Soc. C. E., Vol. II (1873), p. 119, etc.

‡ "On Repairing the Cables of the Allegheny Suspension Bridge at Pittsburgh, U. S. A.", *Minutes of Proceedings,* Inst. C. E., Vol. LXXVI (1883-84), p. 334.

Mr. Collingwood died on August 18th, 1911. He was married on June 5th, 1860, to Eliza W. Bonnett, of New York City, the daughter of Daniel and Margaret Bonnett, who survived him.

He was a member of the American Institute of Mining Engineers, New York Academy of Science, Institution of Civil Engineers, American Geographical Society, Metropolitan Museum of Art, New York Microscopical Society, American Association for the Advancement of Science, and American Institute of Architects.

Mr. Collingwood was elected a Member of the American Society of Civil Engineers on March 3d, 1869. He served as a Director of the Society from 1873 to 1876, and as Secretary from 1891 to 1894. He instituted and endowed the Collingwood Prize for Juniors in 1894.

HARTWELL PRENTICE FARRAR, M. Am. Soc. C. E.*

DIED DECEMBER 16TH, 1921.

Hartwell Prentice Farrar, the son of Josiah P. and Dorcas Whelpley Farrar, was born in Syracuse, N. Y., on July 7th, 1839. His paternal ancestors were early settlers of Connecticut, and his great-grandfather was a Captain in the Revolutionary War. On his mother's side, his ancestors had come from Wales and had settled in Vermont.

Hartwell Prentice Farrar received his early education in the public schools of Syracuse, N. Y., and at Syracuse Institute. He was prepared to enter Rensselaer Polytechnic Institute, but his father's death in 1855 compelled him to go to work, at the age of sixteen, in support of the family. Thus, he began an active life which he continued until 1918, when failing health, following an accident, compelled him to retire, although he never lost interest in engineering and kept in touch with affairs until his death.

Mr. Farrar began his engineering career in 1855 as Chainman and Rodman with the United States Deputy Surveyor on the sub-division of townships on the Upper Mississippi River between Crow Wing (Fort Ripley) and Lake Melacs. He was also engaged on the survey of the La Crosse and Milwaukee Railroad from La Crosse to the Wisconsin River. In 1856 and 1857, he was engaged as Assistant on the preliminary location surveys for the Mississippi and Missouri Railroad (afterward the Chicago, Rock Island, and Pacific Railroad) between Iowa City and Council Bluffs, Iowa, under the late Gen. Grenville M. Dodge, Hon. M. Am. Soc. C. E., Engineer in Charge. In 1857, Mr. Farrar served as Assistant Engineer in charge of a level party on the Illinois River Improvement. He was also in charge of the soundings on this work, under John B. Preston, Chief Engineer, and Joseph W. Trutch, Engineer in Charge, who, later, became Lieutenant Governor of the British possessions at Victoria, B. C., Canada. A report, made in 1863 to the United States Government, on the practicability of making the river and canal a waterway for ships between Lake Michigan and the Mississippi River, was based on this survey.

In 1858, Mr. Farrar was employed as Assistant in charge of a leveling party, with headquarters at Morris, Ill., on changing the levels and locks of the Morris Division of the Illinois and Michigan Canal. He was also engaged on the Fox River Feeder at Dayton, Ill.

In 1859, he went to Colorado Territory where he was engaged in surveying and mining prospecting. He was given a contract to survey and plat the town site of Denver, and had about completed his arrangements to begin the work, when the Pike's Peak mining excitement occurred. He joined the rush to the prospective gold field and gave up the survey of the future city. While in that section of the country, however, he surveyed Boulder City and Arapahoe County, then a part of Kansas, but now in Colorado.

* Memoir prepared by W. S. Caruthers, M. Am. Soc. C. E.

After leaving Colorado, Mr. Farrar settled in Washington County, Illinois, where he was appointed Deputy County Surveyor.

In September, 1861, he enlisted, in Nashville, Ill., in Company D, 48th Regiment of Illinois, for service in the Civil War. He was soon commissioned a First Lieutenant and, later, Captain. He was in action with his regiment at the battle of Fort Donelson and at Shiloh, also known as Pittsburg Landing, Tenn., where he was wounded. He was afterward assigned to duty as Provost Marshal at Memphis, Tenn., and, in 1862-63, was detailed as Engineer Officer in charge of railroad defenses and fortifications on the lines of the Mobile and Ohio and Mississippi Central Railroads. Captain Farrar also acted as Engineer Officer on the staffs of Generals Sullivan, Kimball, Hurlburt, Washburn, and Dana. He served until the close of the war and received his honorable discharge from the Army in the fall of 1865, when he resumed the practice of his profession.

In 1866, Captain Farrar, as he was afterward known, went to St. Paul, Minn., and was engaged for fifteen months as Assistant Engineer on United States river improvements under Gen. G. K. Warren. This work included surveys on the Minnesota River from the Upper Agency to Fort Snelling and on the Mississippi River from St. Anthony to Lansing, Iowa. In 1867 and 1868, he was engaged on the Minnesota River Railroad as Assistant in charge of the work of location and construction, and, in 1868 and 1869, he served with the Pittsburgh, Fort Wayne and Chicago Railroad on railroad accounting, in the office at Allegheny, Pa.

Captain Farrar then gave up the practice of his profession and went into business in Cleveland, Ohio, where for about a year he had charge of the Erie Grain Elevator. Although successful in this work, the call of his profession was too strong and, in the fall of 1870, he went to Newark, Ohio, where he was engaged as Principal Assistant in charge of location and construction on the Newark, Somerset, and Straightsville Railroad. This work comprised 45 miles with heavy rock and tunnel work.

In 1871, he went to Peru, where he was employed on the construction of the Callao, Lima, and Oroya Railroad, commonly known as the "Meiggs" Railroad. He first had charge as Division Engineer of the Lima Division from Lima to San Bartolome, at the foot of the Andes Mountains, a distance of 55 miles. On the completion of this work, he was assigned to the Varugas Division, but while on the latter, he was taken ill with fever and after being in the hospital for several weeks, was granted a leave of absence and returned to his home in Jackson, Tenn., in 1872.

From 1872 to June, 1886, Captain Farrar was in the employ of the Mississippi Central Railroad Company, later re-organized as the Chicago, St. Louis, and New Orleans, which became a part of the Illinois Central System about 1882. In 1872, the main line extended from New Orleans, La., to Jackson, Tenn. In that year, it was decided to build north to Cairo, Ill., and connect with the southern terminus of the Illinois Central. Captain Farrar was employed on this work as Engineer in charge of construction, until the completion of the extension. He was then placed in charge as Engineer of the line from Cairo to New Orleans.

From 1881 to 1886, he was occupied chiefly in the location and construction of branch lines of this Company, under the charter of the Yazoo and Mississippi Valley Railroad Company. He had charge of the line built from Jackson, Miss., through Yazoo City, north to a connection with Memphis, and made many surveys for branches and line revisions and improvements for the Company.

In June, 1886, he resigned his position with the Illinois Central Railroad Company to engage in the construction of railroads as a Contractor. He was so occupied until 1889, when he was appointed Chief Engineer of Maintenance on the Mobile and Ohio Railroad. He remained with this Company until 1900, when he resigned to engage in private practice as a Consulting Engineer.

During the time he was with the Mobile and Ohio Railroad Company, he had charge, as Chief Engineer, of the location and construction of its Montgomery, Ala., line, an important connection about 200 miles in length. He was also engaged on important work, in connection with the Illinois Central Railroad, in an entrance to Cairo, Ill., for the Mobile and Ohio Railroad Company.

Captain Farrar was engaged in private práctice from 1900 to January, 1918, when failing health forced him to retire. During this period, he was very active in his profession, having had charge of surveys and having made reports on many proposed railroads, including the one from Memphis, Tenn., to Pensacola, Fla. He was also occupied with surveys and construction of highways, municipal improvements, and drainage canals, his work taking him into many States of the South and also to Wyoming.

In June, 1921, he was appointed by the Governor of Tennessee as a delegate to the National Drainage Congress held at St. Paul, Minn., but his health would not permit him to serve.

Captain Farrar was married, in 1863, to Miss Agnes Wilson, of Jackson, Tenn., who, with a daughter, Mrs. George Mamer, survives him.

He was a member of the Protestant Episcopal Church for the greater part of his life and for many years was a Vestryman of St. Luke's Church, Jackson. He was a member of the Masonic Order and a Past Grand Commander of the Knights Templar. He was a Charter Member of the Jackson Lodge of Elks. He was always much interested in civic affairs and everything that pertained to the welfare of his community.

Captain Farrar was a man of the highest integrity, was greatly beloved by his friends, and trusted and respected by all with whom he was associated, and no one stood in higher esteem in the community in which he had lived since early manhood. He served his country faithfully and was an honor to his profession.

Captain Farrar was elected a Member of the American Society of Civil Engineers on November 1st, 1893.

HUBBARD MOYLAN FEILD, M. Am. Soc. C. E.*

DIED APRIL 11TH, 1920.

Hubbard Moylan Feild was born in Petersburg, Va., on June 4th, 1872. He was the son of Colonel Everard Meade Feild, a member of an old Virginia family and a veteran of the Confederate Army, in which he served throughout the Civil War. His mother, Maria Louisa (Fox) Feild, was born in Pennsylvania.

Mr. Feild was one of a family of five boys and five girls, being the third eldest of the boys. It is said that he was a real boy, with a strong body and an active mind, much interested in everything pertaining to Indian lore and outdoor life and adventure. He was a leader in his school days and apt in his studies, qualities which acquaintances and comrades of his later life recognized in his maturity.

When he was about eighteen, he received an appointment to the United States Naval Academy, at Annapolis, Md., where he spent about two years. His next experience was on railroad work, in the State of Texas, for the Brazos River Channel and Dock Company, and the Velasco Terminal Railway Company, during which time he took a special course in the University of Texas.

Shortly after this, Mr. Feild began his work in the Tropics, in Costa Rica, Central America, where he was employed as Locating Engineer on the Banana River Railway. It was at this time that the well-known developer, Mr. Minor C. Keith, proposed and made a contract for the sanitation of, and municipal works for, Port Limon, Costa Rica. The execution of this work was entrusted to Mr. Feild, who carried it out in a prompt and efficient manner, exhibiting executive qualities which led to further responsibilities entrusted by individuals and companies undertaking that very difficult task of construction, under tropical conditions. He remained in Costa Rica until 1906, as Superintendent and Maintenance of Way Engineer of the Northern Railway.

Mr. Feild's later enterprises included railway building in the Republic of Panama, Honduras, and Cuba, for the United Fruit Company and its allied corporations. He also had the contract for the sanitation of the Port of Almirante in the Republic of Panama.

The writer is intimately acquainted with most of the work done by Mr. Feild and can state that much of it was carried out in the face of the same difficulties which confronted the builders of the railroad across the Isthmus of Panama. Later years, of course, brought relief from the menace of yellow fever which had inconvenienced his earlier labors.

Mr. Feild had the capacity of enlisting the loyalty of his men to the last degree, a quality without which it is impossible to execute tasks amid the discouragements and natural climatic depressions obtaining in these latitudes. The writer, from his intimate acquaintance with Mr. Feild, would bear testimony to the winning and staunch qualities which were part of him. He

* Memoir prepared by T. Howard Barnes, M. Am. Soc. C. E.

was always ready to lend a hand, or indeed his purse, when a call of deserving need came to him. He never lost his faith in human nature, and his entire career furnishes an example of optimism which may well be a model for his fellow members.

In his later life, Mr. Feild made his home in New Orleans, La., where his house was ever open to his friends. His last days, although clouded with suffering, were made bright by heroic fortitude and the loving care of his family.

He is survived by his wife, who was Miss Edith Sears, of Boston, Mass., a son, and a daughter.

Mr. Feild was elected a Member of the American Society of Civil Engineers on October 1st, 1913.

SAMUEL MERRILL GRAY, M. Am. Soc. C. E.*

DIED NOVEMBER 5TH, 1921.

Samuel Merrill Gray, the youngest son of David and Sarah M. Gray, was born at West Andover, Mass., on November 16th, 1842. His father was engaged in agricultural pursuits at that time, and the boy was brought up, as was customary with farmers' sons, working on the farm and attending the district school.

Later, his father moved from the farm to a part of the town near Phillips Academy, and it was then that the son began laying the foundation for his future, and decided to adopt the Profession of Civil Engineering for his life work. In this decision, he was encouraged by his teachers who recognized his ability and genius in mathematics and kindred subjects. While still in his teens, and striving to enter Harvard Scientific School, his health failed, compelling him to abandon, for the time being, the schoolroom for some outdoor occupation.

His instructors had been watching him for some time with considerable anxiety, and soon saw that if he continued his studies at the Academy, a general breakdown in his health was inevitable. Therefore, they advised him to give up the thought of entering college, and obtain his education in the field. He accepted their advice, gave up his studies early in the course at the Academy, and, in August, 1862, went to sea on a sailing vessel, returning early in the following year, greatly improved in health and fully determined that nothing should turn him from his purpose of becoming a civil engineer, even if he was unable to continue his preparatory course at the Academy.

At that time the engineering business was not brisk, and Mr. Gray experienced considerable difficulty in obtaining a position. Perseverance was one of his strong characteristics, and he finally secured a position in an engineering office at Woonsocket, R. I., where he was employed on what was then the New York and New England Air Line Railroad, now a part of the New York, New Haven, and Hartford Railway System. Later, he was engaged in locating lines on the Connecticut Valley Railroad between New Haven and Hartford Conn., and on several other railway lines in the Middle West.

During these earlier years, while engaged in railroad building, he became interested in hydraulic and municipal work, devoting his spare time to these studies, particularly the designing and building of water-works systems and other municipal works.

Mr. Gray soon acquired a name for himself in his chosen profession, and was early recognized as a rising, ambitious, young engineer. He secured an appointment as an Assistant, and, later, became Chief Engineer of the water supply system of Hartford, retaining this position until the work was completed.

In 1869, Mr. Gray came to Providence, R. I., to accept the position of Assistant Engineer under the late J. Herbert Shedd, M. Am. Soc. C. E., on the water supply system of Providence. He was placed in charge of the con-

* Memoir prepared by George H. Leland, M. Am. Soc. C. E.

struction of the Sockanosset and Hope Reservoirs, and other important parts of the system, remaining with Mr. Shedd until the works were completed.

On February 5th, 1877, he was elected City Engineer of Providence, a position which he held until May 5th, 1890. During his incumbency of this office, he was nominally in charge of all public works, water, sewers, streets, bridges, and harbors.

In 1884, the question of sewage disposal for the City of Providence was being agitated, and Mr. Gray with one of his assistants, the late Charles H. Swan, M. Am. Soc. C. E., was sent to Europe to make an exhaustive and comprehensive study of the various methods of sewage disposal then in use.

As the result of his observations and studies abroad, a report, with recommendations for intercepting sewers and sewage disposal, was submitted by him to the City authorities, and this report and recommendations were approved by Rudolph Hering, M. Am. Soc. C. E., and the late Joseph P. Davis, M. Am. Soc. C. E. The report was recognized at once as a valuable document on the subject of sewage disposal, and it became widely known in the United States and in Europe.

Early in 1890, certain changes in the management of the Public Works Department of the City of Providence made it necessary for Mr. Gray to resign his position as City Engineer, and he opened a private office for the practice of his profession. His first work of importance was the design of a sewerage system and a water-works plant for Chevy Chase, a suburb of Washington, D. C., which was followed by a sewerage system and sewage disposal plant for Chautauqua, N. Y.

His private practice soon increased and he was called to various parts of the country to advise regarding water and sewerage systems, and to act as expert in many damage suits, including the Kettle Brook cases, Blackstone Valley cases, "The Mill Owners *vs.* City of Worcester, Mass., for diverting the waters of Kettle Brook and its tributaries"; also, "The Mill Owners *vs.* City of Woonsocket, R. I., for diverting the waters of Crookfall Brook."

Among the places to which Mr. Gray was called to advise regarding water and sewerage systems were: Augusta and Brunswick, Me.; Meredith, N. H.; Bradford, Vt.; Natick, Springfield, and Worcester, Mass.; East Providence, Bristol, Newport, and Westerly, R. I.; Binghamton, Syracuse, Cohoes, Troy, and Hornell, N. Y.; Lancaster, Philadelphia, and Meadville, Pa.; Wilmington, Del.; Paterson, N. J.; Albuquerque, N. Mex.; Helena, Mont.; Boise, Idaho; and several other places in the South and West, including Havana, Cuba, Toronto, Ont., Canada, and Monterey, Mexico.

He was associated with Mr. Hering on a sewage disposal plant for Baltimore, Md., and with Mr. Hering and the late Frederic P. Stearns, Past-President, Am. Soc. C. E., on a sanitary system for the District of Columbia.

In 1913, there arose the question of an additional water supply for the City of Providence, R. I., and Mr. Gray was chosen as Chief Engineer to the Board of Water Commissioners. This position he retained until the preliminary investigations and surveys to determine the most desirable location for the new source of supply and to secure the necessary legislation for the new project were completed.

In 1915, Mr. Gray resigned the office of Chief Engineer to the Board of Water Commissioners and was appointed Chief Consulting Engineer, a position which he held almost to the time of his death.

Mr. Gray had been a member of the Rhode Island State Board of Health for more than 30 years.

In early life, he became identified with the Christian church, and was a member of the Central Congregational Church of Providence at the time of his death.

Mr. Gray was systematic in his business methods, and required a strict accounting from all his assistants. Although firm, strict, and exacting in all his work, he was never harsh or disagreeable, but always cheerful and kind, and ready to help or assist his students in any way possible.

About 1904, his health became seriously impaired, and it was necessary for him to relinquish his personal practice for a few months in order to recuperate. During this illness, he suffered a great deal, but he bore up under the affliction with fortitude and good nature. Thinking that a change in his environment might prove beneficial, he disposed of his city residence in 1908, and removed to the country. Here, he devoted his spare time and energies to agricultural pursuits and to building a country home for himself at Slocum, R. I. Naturally, he enjoyed country life.

He had made his early home in the vicinity of Hartford, Conn., where he was married to Miss Adelaide S. Gilbert who died in Providence in 1891.

Having been brought up under the shadow of the Andover Theological Seminary, Mr. Gray was a strict observer of the Sabbath. He would permit no work to be done on that day if it was possible to avoid it. It was the writer's pleasure to have been associated with Mr. Gray for the last thirty years as Principal Assistant, and whenever he was sent out on work, his instructions were, that when Saturday night came, if on the journey, to stop off, go to a hotel, and wait until Monday morning, and the bill would be honored.

Mr. Gray firmly believed in justice to all, and was often called on to arbitrate differences which arose between the party for whom the work was being done and the contractor doing the work. He was always glad and willing to aid a contractor in any way compatible with his position as an engineer.

He was always kind and considerate with his students, and took a keen interest in their welfare. He leaves a host of friends who will mourn their loss.

Two children survive him: Mrs. Edith G. Wood, the wife of Dr. Harold B. Wood, Medical Officer of Dodge City, Kans., and Robert Gray, of Slocum, R. I.

Mr. Gray was elected a Member of the American Society of Civil Engineers on May 15th, 1872, and served as Vice-President of the Society in 1892.

PETER CONOVER HAINS, M. Am. Soc. C. E.*

DIED NOVEMBER 7TH, 1921.

Peter Conover Hains was born in Philadelphia, Pa., on July 6th, 1840. He was graduated from the U. S. Military Academy at West Point, N. Y., in June, 1861, and was appointed First Lieutenant in the 2d Artillery. In July, 1862, he was transferred to the Topographical Engineers, which Corps was consolidated with the Corps of Engineers in March, 1863.

Immediately on his graduation, Lieut. Hains entered active service in the field, and served throughout the Civil War, participating in twenty-three battles and actions. He was brevetted Captain for gallant and meritorious services at the Battle of Hanover Court House, on May 27th, 1862; Major for gallant and meritorious services during the siege of Vicksburg, Miss., on July 4th, 1863; and Lieutenant-Colonel for "gallant and meritorious services during the Rebellion", on March 13th, 1865.

After a year's service as Assistant Engineer on the improvement of the Des Moines and Rock Island Rapids of the Mississippi River, Capt. Hains was, in 1867, placed in command of the Engineer Depot and Post of Jefferson Barracks, Missouri. It was his first independent command. The qualities of thoroughness and sound judgment which had brought him distinction during the Civil War, here had a field of action in which the results could be attributed to him personally. The command of the Post carried with it the command of Company E, Battalion of Engineers. After more than three years of training by Capt. Hains, this company was sent to West Point, N. Y., where it came under the writer's command. Its instruction and discipline were of the highest order. It was a model of what an Engineer Company should be, reflecting the greatest credit on the officer who had trained it. The writer did not, at that time, know Capt. Hains personally, but he often thanked him mentally for the work which lightened his own labors so materially.

From 1870 to 1882, Maj. Hains was employed mainly on lighthouse duty, serving as Engineer of the Fifth Lighthouse District from November, 1870, to May, 1874; Engineer of the Sixth Lighthouse District from June, 1871, to May, 1874, and from July, 1879, to August, 1882; and Engineer Secretary of the Lighthouse Board from June, 1874, to July, 1879.

From 1882 to 1891, he was in charge of fortifications and of river and harbor works at Washington, D. C., and its vicinity. The most important work in this District was the reclamation of the Potomac Flats. Entering the up-stream limits of the City of Washington, with a width of less than 1 000 ft., the Potomac River spread out to a width of about 6 000 ft. opposite the middle and lower parts of the city. The channels for navigation were inadequate, and a large area of flats was uncovered at low tide. One of the main sewers of the city ran through these flats, in an open canal. At high tide, the sewage from this canal spread over the adjacent flats, turning them into a pestilential swamp. The remedy was to excavate channels of moderate width for naviga-

* Memoir prepared by O. H. Ernst, Maj.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E.

tion in the river, and use the excavated material for raising the flats. This work was inaugurated by Maj. Hains in 1882. In 1891, when he was called elsewhere, about three-quarters of the 12 000 000 cu. yd. estimated to be necessary had been placed on the flats. About 620 acres of malignant swamp had been transformed into healthful dry land, on which has since been constructed the beautiful Potomac Park. Although the work was finally completed by others, it is, more than is usual with public works, associated with the name of a single officer. The southerly end of Potomac Park has been named "Hains Point" by the War Department, and stands as a monument to his memory, unique and appropriate. A description of this work prepared by Maj. Hains has been published by the Society.*

From November, 1891, to November, 1894, he was in charge of fortifications and river and harbor works, with headquarters at Portland, Me., and from November, 1894, to May, 1895, was Engineer of the Third Lighthouse District, serving at the same time on various Boards of Engineers, of which the most important was the Board to report on the proposed deep-water harbor at San Pedro and Santa Monica Bays, California, in 1892.

From 1895 to 1900, Col. Hains was in charge of fortifications and river and harbor improvements in the District of which Baltimore, Md., was the headquarters, and served as Division Engineer of the Southeast Division, except while temporarily absent in Central America, and while in the field during the Spanish War.

He visited Central America from December, 1897, to March, 1898, as a member of the Commission created by Congress to examine the Nicaragua route for an Isthmian Canal. The construction of a canal on this route had been begun by an American corporation in 1889. This corporation had been driven into bankruptcy by the financial disturbances of 1893, and its plans and estimates had been severely criticized. The Commission of 1897 recommended some radical changes in the plans, especially in the estimates. Its report, however, was not completed when the revival of the Panama plan attracted the attention of Congress, and led to the creation, in 1899, of an Isthmian Canal Commission, to examine and report on all practicable routes for a canal across the Isthmus. Col. Hains was made a member of this Commission, but before it was created, he was called into the field for service in the Spanish War.

He was appointed Brigadier General of Volunteers on May 27th, 1898, and assigned to the command of the 2d Brigade, 1st Division, 1st Army Corps, which he accompanied to Porto Rico, and which he commanded in the skirmishes at Guayama, on August 5th, and Las Palmas on August 8th, 1898. The peace protocol having been signed, he returned to the United States in September, and was mustered out of the Volunteer Service in November, 1898. For his conduct in this campaign, he was recommended by the Commander of the Expedition, Lieut.-Gen. Miles, for promotion to the grade of Major General, but the promotion was not made at that time.

* *Transactions*, Am. Soc. C. E., Vol. XXXI (1894), p. 55.

The Isthmian Canal Commission of 1899 to 1904 made an examination of all the routes across the Isthmus, and an especially thorough one of the two principal routes, *via* Nicaragua and *via* Panama. The members of the Commission made many long journeys together, visiting Paris and the Kiel and Manchester Ship Canals, and passed many weeks in Central America. Their journeys brought them into close personal contact, giving them a more intimate knowledge of each other's character than many years of ordinary intercourse. Travel in Nicaragua especially was made under difficult circumstances. A large part of it was through a wilderness, in which every discomfort had to be endured, and, in the more civilized part, the public accommodations were rarely sufficient for so large a party. Every temptation was offered for the display of selfishness or bad temper, but on no occasion did Gen. Hains lose his equanimity. His sweetness of disposition and generosity of temper will never be forgotten by his colleagues on those expeditions.

The Commission found that the best route for a canal was *via* Panama; it recommended the purchase of the French property and rights, on which it fixed a valuation of $40 000 000; and it submitted a plan for the canal, with estimates of cost, rejecting the sea-level plan and providing for locks. The property was purchased by the United States at the price mentioned, but it was not until 1904 that all legal and diplomatic formalities had been complied with, and the United States was free to begin the work of construction.

For construction purposes, the President decided to re-organize the Commission, and no member of the Engineer Corps of the Army was appointed. At the end of the year the Third Isthmian Canal Commission was organized on which were two officers of the Engineer Corps of the Army, of whom Gen. Hains was one. These officers found themselves in a minority of two on the subject of the type of canal. Shortly before the dismissal of the Second Commission, its Engineering Committee had submitted a report strongly favoring the sea-level type. The full Commission had not had time to take action on the report, but its contents were known to, and approved by, the President and the Secretary of War. Simultaneously with the appointment of the Third Commission, the President appointed a Consulting Board of thirteen distinguished engineers to determine the plan for the canal. This Board was unable to agree, but eight members, including all the foreigners, reported in favor of the sea-level type, whereas five favored the lock type.

The two reports of the Consulting Board came before the Isthmian Canal Commission for review and recommendation. The review of the subject by the Commission satisfied the President and the Secretary of War, and eventually Congress, that the proper type of canal for this Government to build was the lock type.

Gen. Hains had been promoted through the various grades in the Corps of Engineers until April 22d, 1903, when he was appointed Brigadier General in the line of the Army. He was retired from active service on July 6th, 1904, having reached the age of 64, the statutory limit. Under the provision of an Act of Congress, approved August 29th, 1916, he was promoted to the rank of Major General on the Retired List on November 2d, 1916.

When war with Germany was declared in 1917 he offered his services for any duty to which the Government might wish to assign him, and was placed in charge of the Engineering District of which Norfolk, Va., was the headquarters, where he served throughout the World War. He was probably the only officer of the Army who was on active duty throughout the Civil, Spanish, and World Wars.

Gen. Hains was married on November 17th, 1864, to Miss Virginia Pettis Jenkins, daughter of Rear-Admiral Thornton A. Jenkins, U. S. Navy. His widow and three sons survive him.

Those who knew him as the gallant soldier, the skillful engineer, the accomplished gentleman, and the loyal friend, can understand what a stunning blow his death must be to those who knew him also as the tender and devoted protector.

Gen. Hains was elected a Member of the American Society of Civil Engineers on April 2d, 1890.

JOHN BAILLIE HENDERSON, M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1921.

John Baillie Henderson was born in London, England, in 1836. His father was a native of Ross-shire, and his mother of Ayrshire, Scotland. He was educated at Stanmore, Lanark, and Glasgow, and when a young man went to Australia where he began his professional career by work on railroads, roads, and bridges.

In April, 1878, Mr. Henderson went to Queensland and was appointed Resident Engineer on the completion of the Coliban System, then the largest water-works system in Victoria, the cost of which was more than £1 000 000 and from which the mining towns of Taradale, Castlemaine, Chewton, and Bendigo are supplied with water for domestic, manufacturing, and mining purposes. His duties in connection with this work included the revision of the System, which made new surveys and new designs necessary, and these were all executed under his direction. Previous to this work, he had completed the Geelong System, the cost of which was £300 000.

In 1883, Mr. Henderson was appointed Government Hydraulic Engineer of Queensland, and he will always be remembered for his activities in connection with the development of the artesian bore system which made so great a difference in the value of the western lands. He was among the first engineers to give serious consideration to the power possibilities in these released underground waters, and he believed the supply was not inexhaustible. It was largely through his warnings of the danger of exhaustion that the Government assumed control of the sinking of bores. He was also interested in plans for irrigation and stream gauging which had been allotted to his Department.

Mr. Henderson retired from the position of Government Hydraulic Engineer of Queensland at the end of 1916, but, in spite of his eighty-five years, always took an active interest in public affairs.

He died at his home, Monkira, Hawthorne, Brisbane, Queensland, Australia, on February 15th, 1921, after an illness of a few days, and is survived by his wife, two sons, Hector and Ernest Henderson, and one daughter, Mrs. E. H. Pike.

Mr. Henderson was a member of the Institution of Civil Engineers, Institution of Mechanical Engineers, Society of Engineers, Association of Engineers and Shipbuilders of Scotland, Royal Geographical Society of Australasia, Royal Society of Victoria, Institution of Surveyors and Engineers, Victoria, and the Engineering Association of New South Wales.

Mr. Henderson was elected a Member of the American Society of Civil Engineers on June 4th, 1890.

* Memoir compiled from information supplied by E. J. T. Manchester, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

WILLIAM RYAN HILL, M. Am. Soc. C. E.*

DIED JUNE 16TH, 1918.

William Ryan Hill, the seventh son of John J. Hill, was born in New York City, on June 21st, 1854. He received his education at St. John's College, Fordham, N. Y., and Tremont Seminary, Pennsylvania.

On the completion of his education, at the age of 18, Mr. Hill immediately obtained a position with the New York Central and Hudson River Railroad Company, and advanced so rapidly that in two years he was placed in charge of the elimination of the grade crossings on Fourth Avenue from 115th Street to the Harlem River, in New York City.

For a number of years, Mr. Hill was engaged on the construction of the Elevated Railroad in New York City. He then entered the service of the Boston, Hoosac Tunnel, and Western Railway Company as Division Engineer in charge of construction. On the organization of the South Pennsylvania Railroad Company, Mr. Hill was appointed Senior Assistant Engineer and, for three years, he was engaged in making extensive topographic and location surveys in the Alleghany Mountains of Pennsylvania. The surveys made by the Engineering Corps of the South Pennsylvania Railroad Company for the determination of the final location of the main line, were considered as models for many years.

In 1889, he was appointed Chief Engineer of the Syracuse Water-Works, and under his supervision the new system of water supply for that city was constructed and operated. It is acknowledged to be one of the most perfect in the United States in all its details, costing more than $3 000 000, yet the aggregate of all payments to the contractors, including extra bills, was within 1% of Mr. Hill's estimate on which the contracts were based. In 1895, the Society awarded him the Thomas Fitch Rowland Prize for a paper† describing the engineering features of this work.

On December 31st, 1899, much to the regret of the officials of the City of Syracuse, he resigned his position to accept the appointment of Chief Engineer to the Aqueduct Commission of New York City. During Mr. Hill's occupancy of this position, plans were perfected for the New Croton Dam, the Jerome Park Reservoir, and other structures of magnitude, on which both the engineering and construction work was dispatched with marked ability. During the period of this duty, his health failed, and he resigned in July, 1903. In accepting his resignation, the Aqueduct Commission complimented him on his zeal, courage, and engineering skill in so successfully carrying on this great work.

On account of poor health, Mr. Hill accepted no active engagements, serving only in an advisory capacity until January 1st, 1907, when he was appointed Special Deputy State Engineer of New York and placed in charge

* Memoir prepared by Frank Sutton, M. Am. Soc. C. E.

† "The Water-Works of Syracuse, N. Y.", *Transactions*, Am. Soc. C. E., Vol. XXXIV (1895), p. 23.

of the construction of the Barge Canal during 1907 and 1908. At this time, he was offered the nomination for State Engineer, but declined it, as he preferred not to enter the political field.

In 1909, he opened an independent office in Albany, N. Y., and soon acquired a large and important consulting practice which he continued until his death, which occurred at his home there on June 16th, 1918.

In July, 1874, Mr. Hill was married to Miss Bella S. Baxter, of White Plains, N. Y., who survives him. He is also survived by four sons, John J., Joseph B., Edward B., and William R., Jr., and one daughter, Miss Alice R. Hill.

Mr. Hill was a splendid example of American manhood, clean cut, able, persistent, and industrious, and by his fairness, kindness, and generosity, he built himself deep into the affections of a myriad of people who felt proud to call him friend. His fondness for and devotion to his home and family were notable traits in his character.

His advice for success was "that no one can accomplish great things unless he aims at great things and pursues that aim with untiring energy, unflinching courage, and self-confidence."

At one time he was President of the American Water Works Association and of the Albany Society of Civil Engineers. He was also a member of the New England and Central States Water Works Associations.

Mr. Hill was elected a Member of the American Society of Civil Engineers in July, 1891, and was a Director of the Society at the time of his death.

HOWARD CARLETON HOLMES, M. Am. Soc. C. E.*

DIED OCTOBER 30TH, 1921.

Howard Carleton Holmes was the only son of Cornelius and Maria Folger Holmes. He was born on the Island of Nantucket, Massachusetts, on June 10th, 1854, and when five years of age went with his parents to San Francisco, Cal.

After receiving his education in the public schools of San Francisco, he entered the City Engineer's Office of Oakland, Calif. Later, he began as a Surveyor and became identified with a number of leading engineers. Mr. Holmes was only 19 years old when he made the topographic surveys and maps necessary for the development of Lake Chabot, the principal source of water supply of the City of Oakland. At 21, he passed an examination for appointment as United States Deputy Surveyor, and soon afterward became Assistant Engineer of the Board of State Harbor Commissioners, leaving this position to design and build the Alameda Mole and Depot for the South Pacific Coast Railway Company.

Mr. Holmes then directed his attention to street railway construction and, in 1887-88, designed and built the Powell Street Cable Railway, then known as the Ferries and Cliff House Railroad. During the next few years, he designed and built the cable railroads at Portland, Ore., at Spokane, Wash., and the Madison Street Railway in Seattle, Wash. Returning to San Francisco, he designed and constructed the Sacramento Street Branch of the Ferries and Cliff House Railroad and the lower end of the California Street Railroad, and extended the Union Street Cable Railroad from Fillmore Street to the Presidio. Later, he secured the contract for the design and construction of the original electric railway at Stockton, Calif.

In 1892, Mr. Holmes was appointed Chief Engineer of the Board of State Harbor Commissioners and many wharf structures along the water-front of San Francisco and also the Union Ferry Building at the foot of Market Street, the foundation of which was a notable piece of engineering work, were built under his direction. He also designed the passenger and freight ferry slips of all the railroads on San Francisco Bay, except those of the Southern Pacific Company. Mr. Holmes was the designer and inventor of many improvements and innovations in harbor construction.

In 1901, he resigned his position with the State Harbor Board to become Chief Engineer of the San Francisco Dry Dock Company to design and construct a 750-ft. granite and concrete graving dock at Hunter's Point. In 1916, as Consulting Engineer for the Bethlehem Shipbuilding Company, he designed and supervised the construction of a 1 025-ft. concrete and granite graving dock also located at Hunter's Point, San Francisco.

From 1902 to 1910, he was retained as Chief Engineer of San Francisco-Oakland Terminal Railways, and designed and supervised the construction of ferry slips, terminal buildings, wharves and extensive double-track trestle. From 1905 to 1910, he was also retained as Consulting Engineer during the

* Memoir prepared by James J. Walsh, Assoc. M. Am. Soc. C. E.

construction period of the Western Pacific Railroad, and designed and supervised the Oakland Freight and Passenger Terminal and the San Francisco Freight Terminals of that Company. Mr. Holmes was also engaged by the Panama-Pacific International Exposition from 1913 to 1915 as Consulting Engineer. In addition, at different periods during the last decade, he was retained in the capacity of Consulting Engineer for the Esquimalt Shipbuilding and Drydock Company, Moore Shipbuilding Company, Richmond Belt Railway, South San Francisco Land and Improvement Company, Northwestern Pacific Railway Company, and others.

His judgment and intelligence gave great weight to his opinions in all matters pertaining to engineering work. Mr. Holmes was eminently successful in his chosen profession; his business dealings were characterized by the utmost fairness to his clients and contractors alike, and a host of friends along the entire Pacific Coast mourn his loss. His death, which occurred on October 30th, 1921, resulted from a stroke of paralysis after an illness lasting several weeks.

In 1883, Mr. Holmes was married to Miss Josephine Bauer who survives him.

He was a member of the Engineers' Club, the California Academy of Science, and the Seismological Society of America, and a prominent Mason and Knight Templar. Mr. Holmes was also prominent in the club life of San Francisco, having been a member of the Pacific Union, Bohemian, and San Francisco Golf and Country Clubs.

Mr. Holmes was elected a Member of the American Society of Civil Engineers on November 4th, 1903. He was also a member of the San Francisco Section of the Society.

ISAAC WENDELL HUBBARD, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1920.

Isaac Wendell Hubbard, the son of Mark C. and Marian (Wendell) Hubbard, was born on October 24th, 1872, in Greensboro, N. C. He was educated at the public schools of Philadelphia, Pa., Pennington Seminary, and by private tutors.

In 1890, he entered the service of the City of Philadelphia, in the Bureau of Surveys, where he remained for ten years under the late Samuel L. Smedley, M. Am. Soc. C. E., and his successor, George S. Webster, Past-President, Am. Soc. C. E., Chief Engineer. During this time he was engaged on property surveys, on work in connection with the piers and abutments of the Philadelphia and Reading Elevated Railroad, and on the Pennsylvania Avenue Subway and Tunnel. On the completion of the Pennsylvania Avenue Subway, Mr. Hubbard was assigned to the work of making hydrographic surveys of the Delaware and Schuylkill Rivers, removing Schooner Ledge Rock, and other work in connection with port and harbor improvement.

In 1900, he became a partner of Mr. J. Orie Clarke, under the firm name of Clarke and Hubbard. The firm conducted a general engineering practice for three years, when the partnership was dissolved and Mr. Hubbard became for a time Engineer of Construction for Ryan and Kelley on the Low Grade Freight Line of the Pennsylvania Railroad at Herrville, Pa. This was a heavy piece of work, involving 80-ft. embankments and masonry bridge construction, and Mr. Hubbard had about 800 men under his direction. On the completion of this contract, in the early part of 1904, he became a partner of Marshall R. Pugh, M. Am. Soc. C. E., under the firm name of Pugh and Hubbard, Civil and Sanitary Engineers, the partnership continuing until the outbreak of the World War in 1917. At that time, the senior partner went to France as Major of Engineers, and Mr. Hubbard, after a short period as Superintendent of the Emmons Coal Mining Company, entered the service of the Government as Senior Engineer, Division of Shipyard Plants, U. S. Shipping Board Emergency Fleet Corporation.

During the period from 1900 to 1917, Mr. Hubbard was engaged on the surveys, location, and supervision of roads at Valley Forge Park, Pennsylvania, for the Valley Forge Park Commission; factory building (slow-burning construction) for the H. O. Wilbur Company; surveys, plans, and supervision of a Licorice Manufacturing Plant, at Camden, N. J., which included dredging, bulkheads, wharves, and factory buildings; topographical surveys and development of extensive tracts of land on Long Island and elsewhere; and engineering and location of a great number of interurban trolleys in New Jersey, Pennsylvania, and Ohio.

Becoming interested in contracting and, subsequently, in quarrying, he had charge of a large number of important pieces of work. He was President of

* Memoir prepared by Marshall R. Pugh, M. Am. Soc. C. E., and Howard D. Campbell, Assoc. M. Am. Soc. C. E., a Committee of the Philadelphia Section of the American Society of Civil Engineers.

the Main Line Stone Company, which operated several quarries and engaged in a general contracting business. Real estate development, construction of roads, concrete, water-bound, and bituminous macadam highways, trolley lines, water-works, and sewers, were among his activities.

Mr. Hubbard's duties with the U. S. Shipping Board Emergency Fleet Corporation were administrative, and covered the supervision of plant improvements financed by the Fleet Corporation; the review of proposed plant extensions; the amount of money to be expended on such improvements; the policies to be followed; and, finally, the allocation of concrete shipyards.

Leaving the employ of the Government in April, 1920, he became Engineer for the Union Petroleum Company, having charge of the plant construction at Clarendon, Pa., involving an expenditure of $500 000, and at Marcus Hook, where a lubricating plant was erected, consisting of concrete buildings, 525 ft. of bulkhead, and a large amount of tankage, involving an expenditure of $1 000 000. He organized the force and had entire charge of the work at both these places, and also at a plant for the same company at New Orleans, La., costing $500 000.

Mr. Hubbard was a man of tremendous energy, and whatever he undertook he pushed with the greatest vigor. He was a member of St. Paul's Presbyterian Church, West Philadelphia, Pa., and was very active in church affairs. He was also a member of Conrad B. Day Lodge No. 645, F. and A. M., and of St. John's Chapter No. 232, Royal Arch Masons, and of the Engineers' Club of Philadelphia.

On October 5th, 1899, Mr. Hubbard was married to Miss Cecelia McCorkell, who survives him. He also leaves two children, a son and a daughter.

Mr. Hubbard was elected an Associate Member of the American Society of Civil Engineers, on March 1st, 1905, and a Member on January 4th, 1910.

WILLIAM TYNDALE JENNINGS, M. Am. Soc. C. E.*

DIED OCTOBER 4TH, 1906.

William Tyndale Jennings was born at Toronto, Ont., Canada, on May 19th, 1846. He received his education at the Model Grammar School and at the Upper Canada College in Toronto.

Mr. Jennings began his engineering work in 1869 as Assistant on drainage improvements of the Crown Lands of Ontario, under the late T. N. Molesworth, Engineer with the Ontario Government.

In 1870, he entered the service of the Great Western Railway Company and was engaged in surveying, location, and construction work until 1873, when he was appointed Resident Engineer on Construction of the Air Line Division of that road. Later, he was appointed Resident Engineer of the London and Port Stanley Railway Company and of the Welland Railway Company.

In 1875, Mr. Jennings was made Locating Engineer on the British Columbia Section of the Canadian Pacific Railway, in which capacity, and in addition to other work, he located the present line of that railway through the canyons of the Fraser River. In 1879, he was made District Engineer on the construction of "Section B" of the Canadian Pacific Railway, from Rat Portage to Eagle River, with headquarters at Rat Portage, Ont. On the completion of this work, Mr. Jennings returned to British Columbia as Superintending Engineer on that Section for the Dominion Government.

In 1883, after a year in the East, Mr. Jennings returned to British Columbia as Chief Engineer and Acting Manager of the Construction Company in charge of work on 350 miles of that Section, east from the Pacific Coast, with headquarters at Yale, B. C.

In 1885, he was appointed Chief Engineer of the Detroit Extension of the Canadian Pacific Railway, comprising the section between Woodstock and Windsor, Ont. On the completion of this work which included the construction of the Toronto Wharves from Yonge Street westward, Mr. Jennings was made City Engineer of Toronto, which position he held for one year. During this time, he re-organized the Engineer Department effecting many municipal improvements, among which were the Sherburne Street Bridge and the King Street Subway.

In 1891, he opened an office in Toronto and began private practice as a Consulting Engineer. As Chief Engineer, from 1892 to 1895, he located the line of the "Crow's Nest" Railway. He also had charge of many large projects, among which may be mentioned the construction of electric railway lines, examinations relative to the opening up of vast areas of country to the north and west, for the Canadian Government, and the navigability of various rivers. He continued his private practice until his death on October 4th, 1906.

Mr. Jennings was a member of the Institution of Civil Engineers of Great Britain, of the British Association for the Advancement of Science, the Im-

* Memoir prepared by James H. Kennedy, M. Am. Soc. C. E.

perial Institute, and the Canadian Society of Civil Engineers, of which he was President in 1899. He served for several years as Examiner in Civil Engineering subjects for the University of Toronto and was always enthusiastic for the advancement of engineering education.

He was married in 1876 to Miss McKay, daughter of the Registrar of the County of Elgin, who died in 1887. He was survived by one son, Gordon T. Jennings.

Mr. Jennings was elected a Member of the American Society of Civil Engineers on April 2d, 1884.

SIR JOHN KENNEDY, M. Am. Soc. C. E.*

DIED OCTOBER 25TH, 1921.

Sir John Kennedy was born at Spencerville, Ont., Canada, on September 23d, 1838, the son of William Kennedy of Dumfriesshire, Scotland. At the age of sixteen, he came to Montreal, Que., Canada, and entered the employ of the late Thomas C. Keefer, Past-President, Am. Soc. C. E., who was constructing the first system of water-works for the city. It was in this practical manner, together with supplemental study, that the boy obtained his knowledge of engineering, but he steadily advanced in his Profession until he became one of its leading authorities on this Continent and was also recognized by the Engineering fraternity abroad. He served as Chief Engineer of the Great Western Railway System, and from 1875 until 1907 as the Chief Engineer of the Montreal Harbor Commission. From 1907, until his death, he was the Consulting Engineer of that body. He was also a member of many important Royal Commissions and was widely consulted in engineering matters.

He was knighted on New Year's Day, 1916, in recognition of his distinguished services, and in March, 1917, McGill University conferred on him the degree of LL. D. In May, 1921, he received the degree of D. C. L. from McMaster University, and these are only a few of the many honors bestowed on him in recognition of his marked ability and sterling qualities.

Sir John was married in 1865 to Miss Louise Scott, of Montreal, and Lady Kennedy, together with two daughters, Lady Ames, of Geneva, Switzerland, and Mrs. H. H. Kennedy, of Montreal, survives him.

At the funeral service, his pastor, the Rev. F. L. Orchard, paid the following tribute of respect and love to his memory and to his greatness of mind and simplicity of character during a lifetime of 83 years:

"There is, as a great scholar has pointed out, a philosophy of history. As we spread the broad expanse of centuries before the mind, strong leaders stand out here and there like great rocks to relieve the dreary wastes and in the shadow of their strength and courage the multitudes have been sheltered from the storms of superstition, vice, and ignorance. There is a sense in which all humanity rests in the shadow of its great lives. * * *

"The natural thing to say first in a tribute to the life of Sir John Kennedy is that he was a genius in the realm of Engineering. The divine and human factors that entered into the endowment of his mind, made him an easy master of the intricate tasks and problems of that calling. It was his playful boast that he began his engineering career at the age of thirteen and indeed he did, at that age, run the engine of his father's mill at Aylmer. He was not a graduate of the schools or universities. Knowledge is acquired, but genius is a gift, and Sir John Kennedy was a genius. Despite his modesty, he recognized his gifts, he regarded them as a holy trust, and from first to last, he sought to use them in the interests of his fellow men. How well he succeeded is indicated by the opening words of an editorial that appeared in yesterday's press: 'Sir John Kennedy is dead, but an imperishable monument

* Memoir prepared by Frederick W. Cowie, M. Am. Soc. C. E.

remains—the Harbor of Montreal. To him more than to any other man, the country owes its great National harbor'. By his death Montreal lost one of her most widely known and most greatly loved and honored citizens."

At the meeting of the Engineering Institute of Canada on October 27th, 1921, the writer offered the following remarks:

"There are few who have had the opportunity of so intimate and long an engineering or professional connection with the late Sir John Kennedy as the speaker.

"Thirty-eight years ago, it was my good fortune to join the Staff of the Harbor Commissioners, whose Chief Engineer was John Kennedy. During the whole of the thirty-eight years, in connection with the improvements to the River St. Lawrence Ship Channel, the development of public harbors in Canada, and especially the development of the Port of Montreal, it has been my good fortune to be associated with the great Engineer who has to-day been laid to rest.

"In 1886, it was no small thing to be employed under John Kennedy. Until the day of his death, it was no small matter to be professionally associated with the great consulting engineer, whose loss will be a loss to this city and to this country.

"Sir John Kennedy was eminently an engineer. By personality, by courage, by industry, and by skill, he combined all the essentials of that great Profession.

"When it was first my privilege to know him, he was a profound student of engineering, and until the very last he still continued to be so. Many professional men have received inspiration from Sir John Kennedy, and now that he is gone, he has left a monument of integrity, skill, and success, which will be an inspiration to all, and especially to the members of this Institution."

He also proposed the following resolution:

"Resolved: That the Chairman, Executive and Members of the Engineering Institute of Canada deeply mourn the death of Sir John Kennedy, but that they feel consolation in the thought that their late friend and confrère was spared to a ripe old age, to give the benefit of his sound advice and counsel to the Institute, as well as to the members personally. The Montreal Branch of the Institute is desirous of putting on perpetual record some of the notable acts of its late friend, both in his public and private life, so that his career will be an inspiration to those who come hereafter:

"Sir John Kennedy was eminent as an engineer, both on the Continent of America and abroad, and his sound professional judgment was invoked on many occasions and his suggestions carried out to a successful issue. In the field of hydraulic engineering, mechanical, and construction problems, his genius was outstanding.

"Notable among his successes was the deepening of the River St. Lawrence and the development of the Port of Montreal, under the Harbor Commissioners. The manner in which he carried out the improvements to the River St. Lawrence Ship Channel is outstanding, and the methods he adopted and the mechanical means to that end are still, in many particulars, followed by his successors.

"In paying this last tribute to our departed friend and counsellor, we cannot forget the part he took at the inception of this Society, and the constant attention he gave to its problems and welfare.

"As a citizen he was beyond criticism, and although he took no active part in civic government, his advice was often sought and freely given. His Christian character is without reproach, and his domestic relations were of the happiest.

"The members of the Montreal Branch of the Institute offer their heartfelt sympathy to the family and relatives, and pray that consolation will be afforded to them by dwelling on the record left by him after a long and successful career. To the partner of his life's joys and sorrows, we especially tender our sympathy, and trust that Lady Kennedy will yet be long spared to be a comfort to her bereaved family."

Sir John Kennedy was elected a Member of the American Society of Civil Engineers on September 1st, 1875, and served as a Director of the Society from 1898 to 1900.

EUGENE WILLETT VAN COURT LUCAS, M. Am. Soc. C. E.*

DIED MARCH 8TH, 1921.

Eugene Willett Van Court Lucas was born at Mount Vernon, N. Y., on December 21st, 1864. He was appointed to the United States Military Academy in 1883 and was graduated therefrom in 1887. Lieut. Lucas was first assigned to the Artillery, but, in 1888, was transferred to the Engineer Corps of the Army.

After spending two years (1888-90), at the Engineer School of Application, at Willets Point, N. Y., he was detailed as Assistant Instructor in Practical Military Engineering at the U. S. Military Academy until 1892. From 1892 to 1898, he had charge of various engineering works having to do with rivers and harbors and fortifications in North and South Carolina and at Willets Point, N. Y. As Major, U. S. Volunteers, during the war with Spain, he served as Chief Engineer of the 2d Division, 4th Army Corps, in the camps at Tampa and Fernandina, Fla.; as Chief Engineer of the 3d Division, 2d Army Corps, in the camp at Athens, Ga.; and as Chief Engineer of the 2d Army, at Greenville, S. C. Maj. Lucas was honorably discharged from the Volunteer Service on March 2d, 1899.

On his return to his work with the Corps of Engineers, U. S. Army, he was detailed in charge of the North Carolina Fortification and River and Harbor District, with headquarters at Wilmington, N. C., and on the Mississippi River improvements until January 1st, 1906, when he resigned from the service.

After his resignation from the Army, Maj. Lucas became interested in hydraulic projects in the South. Early in 1913, in connection with the late Col. Thomas W. Symonds, U. S. A., he was appointed by Governor Sulzer as Consulting Engineer for the State of New York on the construction of the State Barge Canal, and continued on this work until 1915. In the turmoil that developed in the State and continued through the term of office of Governor Sulzer, which term ended in his impeachment and removal from office, Col. Lucas performed the duties of his position with marked engineering ability, entirely devoid of the slightest political bias; under the circumstances this was no easy task and will ever remain to his credit as showing that, in the conduct of his work, he operated solely from the point of view of an engineer responsible to the public, as opposed to the exploitation of the State for personal gains.

In 1912, he was appointed Chief of Engineers of the New York State National Guard, and, later, was assigned to the command of the Twenty-second Engineers. In 1916, Col. Lucas went to the Mexican border with this regiment and following its return to New York, he retired from the State service.

In 1917, when the United States entered the World War, Col. Lucas again entered the service—a volunteer—as Lieutenant Colonel of the 304th Engi-

* Memoir prepared by the late John A. Bensel, Past-President, Am. Soc. C. E.

neers. He was commissioned as Colonel of the 66th Engineers and assigned to command Camp Laurel, Maryland, in 1918. He was relieved from this duty and ordered to a United States Hospital where he remained until the cessation of hostilities.

Col. Lucas did not resume the active practice of his profession after the World War; he had become interested in some projects in the Southern States which ended rather disastrously, as far as his personal interests were concerned, and cast a cloud over the last year of his life. In this trouble, he had the sympathy of those who were permitted within a very reserved exterior. His life was one of accomplishment, both as a military and as a civil engineer, and in his death, which occurred on March 8th, 1921, his professional friends will miss one who throughout his life acted with a high conception of his duty as an engineer and a gentleman.

Col. Lucas is survived by two sons, E. W. Van C. Lucas, Jr., and John D. Lucas. His wife, who was Miss Agnes Daniel, of Wilmington, N. C., died on January 29th, 1916.

Col. Lucas was elected a Member of the American Society of Civil Engineers on April 3d, 1895.

WILLIAM LUDLOW, M. Am. Soc. C. E.*

DIED AUGUST 30TH, 1901.

William Ludlow, the second son of William Handy Ludlow and Frances Louisa (Nicoll) Ludlow, was born at Riverside, Islip, Long Island, N. Y., on November 27th, 1843. He came from distinguished ancestry, having been a direct descendant of Roger Ludlow who was appointed by Oliver Cromwell as Lieutenant Governor, in turn, of Massachusetts and Connecticut, and the first of his family to settle in America. His great-grandfather was an Aide on General Washington's staff, and his father served during the Civil War, having been mustered out with the brevets of Brigadier General and Major General, and subsequently was Speaker of the Assembly of the State of New York. His mother was descended from William Nicoll, who settled at Islip on land granted by Charles II in 1683, and who was the first Royal Secretary of the Colony after its transfer by the Dutch.

William Ludlow received his early education at home, having been tutored by the Rev. Henry M. Davis, the rector of St. John's Protestant Episcopal Church at Islip. In 1853, he was sent to Burlington Academy, Burlington, N. J., and, later, to the University of the City of New York, where he was the recipient of the scholarship presented to the family in recognition of the services of his grandfather, Ezra Ludlow, Architect of the University building. In 1860, he entered the United States Military Academy, at West Point, N. Y., from which he was graduated, eighth in his class, on June 13th, 1864.

In 1864, the Federal Government was doing its utmost to bring the Civil War to a close, and the members of the graduating class at West Point were immediately commissioned and sent to the front. Cadet Ludlow received the rank of First Lieutenant, Corps of Engineers, and after a short stay in Washington, D. C., was ordered to report to the Chief Engineer of the Department of Mississippi. He was assigned to the Army of the Cumberland, in which he served as Chief Engineer of the Twentieth Corps until September, 1864. In this capacity, he was engaged in the construction of bridges, the selection of offensive and defensive positions, the design and construction of temporary fortifications, etc. He also took part in the Battle of Peach Tree Creek and was recommended by Gen. Hooker for promotion to brevet rank, "for gallant and meritorious services in laying a bridge across Peach Tree Creek under a severe fire, * * *". He also took part in the battles of the Atlanta Campaign, notably the defense of Allatoona Pass, Ga., on October 5th, 1864, for which he was promoted to the rank of Brevet Captain.

From November, 1864, to March, 1865, Capt. Ludlow served as Chief Engineer of the Army of Georgia on its march to the sea and through the Carolinas. He fought in the battles of Averysborough and Bentonville, the occupation of Goldsborough, and the capture of Raleigh, for which he received commissions as Brevet Major and Brevet Lieutenant Colonel, respectively.

* Memoir compiled from information supplied by Maj. Gen. William M. Black, U. S. A. (*Retired*), M. Am. Soc. C. E., and on file at the Headquarters of the Society.

After the close of Sherman's Campaign, and a leave of absence, he was ordered, in November, 1865, to Jefferson Barracks, Mo., where he organized the Engineer Depot and the newly authorized Company E of the Battalion of Engineers. He remained in command of the Depot and troops until November, 1867, having, in the meantime, received his commission as Captain in the Corps of Engineers. In December, 1867, he was ordered to report to the late Gen. Q. A. Gillmore, U. S. A., M. Am. Soc. C. E. (then Major, Corps of Engineers, U. S. A.) for duty as his Assistant and served under him at Staten Island, New York, and Charleston, S. C., until November, 1872. While Col. Ludlow was detailed on this duty, he caused the steamer *Henry Burden* to be fitted up for pump dredging; this steamer was the first of a class of hydraulic dredges now so successfully used in river and harbor improvement work.

In November, 1872, Col. Ludlow was appointed Chief Engineer of the Department of Dakota, with headquarters at St. Paul, Minn., which appointment he held until May, 1876. In this position, his most important work was his explorations and surveys of the Yellowstone River and through Yellowstone Park and the Black Hills. He was assisted in this work by scientists from various universities who volunteered their services, and the reports of their work went far to bring to the attention of the American people the value and resources of these regions. Col. Ludlow recommended that the care of Yellowstone Park be transferred to the War Department and that it be improved by roads and bridges and opened to the public, which recommendations have since been carried out.

In 1876, Col. Ludlow was ordered to Philadelphia, Pa., as Assistant on river and harbor improvements. He remained in this District until August, 1882, having been promoted to the rank of Major, Corps of Engineers, U. S. A., on June 30th, 1882. During the latter part of this detail he was practically in charge of the work there. Among other things, he improved the methods of the work being done and made such a thorough and comprehensive survey for the improvement of all the navigable waterways that when he was ordered elsewhere a strong effort was made by the citizens of Philadelphia to have him retained in the District. Before he left, he was presented with a memorial signed by the municipal authorities and by the heads of the great commercial, maritime, and railroad interests of the city, "to make an enduring record of their high appreciation of the services of Colonel William Ludlow."

Col. Ludlow's next appointment was as Engineer Secretary to the Lighthouse Board, with headquarters at Washington, D. C. He was retained on this detail until March, 1883, when he was granted a special leave of absence (by Act of Congress February 28th, 1883) to accept the position of Chief Engineer of the Philadelphia Water Department until April, 1886. Col. Ludlow re-organized the Department, correcting the existing conditions, and brought it to an efficient working organization. He had also investigated the question of a water supply for the city and was consulted on that matter from time to time until his death.

In April, 1886, Col. Ludlow was appointed Engineer Commissioner of the District of Columbia which position he held until January, 1888. The experi-

ence gained by his connection with the municipal works of Philadelphia assisted him greatly in this work which was undertaken and carried through with his characteristic energy. Among the many difficult problems solved during his incumbency was the extension of Massachusetts Avenue to the west across Rock Creek Valley, which was severely criticized at the time, but which has since proven his wisdom.

After a short tour of duty in Philadelphia as Engineer of the Fourth Lighthouse District, Col. Ludlow was sent to Western Michigan in December, 1888, to take charge of the river and harbor improvement in that part of the State, with headquarters at Detroit, which detail he held until November, 1893. He also served during part of this time as Engineer of the Ninth and Eleventh Lighthouse Districts of the Great Lakes, and had charge of the river and harbor improvements on the eastern coast of Michigan and the waters connecting the Great Lakes. While in charge of this work, he prepared and put into service the project for lighting the narrow and difficult channels between Lakes Superior and Huron, in which, until this time, it had been customary to stop navigation at nightfall to the great detriment of the lake commerce. On his relief from this duty, Col. Ludlow was presented with a set of resolutions by the Lake Carriers' Association of Cleveland, Ohio, and the municipal and commercial authorities of Grand Rapids, Mich., in recognition of his services.

From November, 1893, to April, 1896, Col. Ludlow served as Military Attaché to the United States Embassy at London, England, having been promoted to the rank of Lieutenant Colonel, Corps of Engineers, in 1895. During this detail, he made an inspection tour of the Suez, Corinth, and Kiel Canals, as well as the maritime canals in Holland.

In April, 1895, Col. Ludlow was commissioned as Chairman of the Nicaragua Canal Board on which he had associated with him Mordecai T. Endicott, Past-President, Am. Soc. C. E., and the late Alfred Noble, Past-President Am. Soc. C. E. This Board was appointed to inspect the route of the proposed Nicaragua Canal and report to President Cleveland on the cost thereon before November, 1895. The Board reported adversely on the proposed cost, and in April, 1896, Col. Ludlow, as Chairman, was recalled from London, where he had returned, to testify in regard to his findings before a Congressional Committee. The report was accepted on his testimony and made the basis for future action on questions relative to the Nicaragua Canal.

Col. Ludlow was next assigned to duty as Engineer of the Third Lighthouse District, with headquarters on Staten Island, New York; but, in 1897, he was transferred to New York City in charge of fortification and river and harbor work which included the improvement of the entrance of New York Harbor. He submitted a report, in 1898, advocating the opening of a deep straight channel from the ocean by the removal of the bar of the so-called East Channel. This project was subsequently adopted and carried out, and, as the Ambrose Channel, now forms the main entrance to the Port of New York.

When war with Spain was declared, Col. Ludlow was ordered to duty as Chief Engineer on the Staff of the Major General commanding. On May

4th, 1898, he was appointed Brigadier General of Volunteers and was sent to Tampa, Fla., as Chief Engineer (temporarily) of the United States forces there under the command of Gen. Shafter, in which capacity he organized the engineer equipment of the Fifth Army Corps. He left Tampa with the Santiago Expedition on June 14th, and on his arrival off Santiago, supervised the transfer of the Cuban Army to Siboney and was assigned to command the First Brigade, Second Division, of the Fifth Army Corps, under Gen. Lawton. The First Brigade took part in the attack on El Caney and Gen. Ludlow's services were highly commended in Gen. Lawton's report on this action. The First Brigade was then moved to Santiago and took a prominent part in that campaign, and Gen. Ludlow was again commended in Gen. Lawton's report.

After the surrender of the Spanish forces, Gen. Ludlow returned to Montauk Point, Long Island, with the First Brigade. In September, 1898, he was commissioned Major General of Volunteers and appointed President of the Board of Officers to make regulations for the transport service. In October, he was assigned to the command of the Second Division, First Army Corps, at Columbus, Ga., where he remained until December, when he was appointed Military Governor of Havana, Cuba, by direction of President McKinley. Gen. Ludlow's orders with this appointment included the charge of "all that relates to the collection and disbursement of the revenues of the port and city and its police, sanitation, and general government"; later, however, he was relieved of that part relating to the collection of revenues. His success as Military Governor was made apparent by the maintenance of order, the cleansing of the city, the organization of the municipal government along new lines, and the reform in the Courts, schools, sanitary conditions, etc. In April, 1899, he was honorably discharged from the Volunteer Service and immediately re-commissioned as Brigadier General, Volunteers, under the new law.

In January, 1900, Gen. Ludlow was appointed Brigadier General, United States Army, and in May of that year, the Department of Havana having been discontinued, he was ordered to return to the United States as President of the War College Board, with headquarters at Washington, D. C. In this capacity he inspected the French and German military establishments and reported thereon and also on needed reforms in the American establishment; at the same time, he presented a project for the proposed War College.

After the completion of his work on the War College Board, Gen. Ludlow was ordered to active service in the Philippines. Before his departure he was honored by being placed in command of the regular army troops participating in the parade at the inauguration of President McKinley on March 4th, 1901.

During the hardships of the Santiago Campaign, Gen. Ludlow's health was badly impaired, and thereafter he suffered from frequent attacks of bronchial trouble. On his arrival in the Philippines, this trouble became worse, and in May, 1901, he was ordered home on the surgeon's certificate of disability. He had never regarded his illness as serious, and this order was obeyed most unwillingly on his part. On his return to the United States, he was taken to his daughter's home at Convent Station, N. J., where he died on August 30th,

1901. He was buried from Trinity Church, New York City, on September 3d, 1901, with military honors, and his ashes were laid in the family burial ground at Islip, Long Island.

Gen. Ludlow was married in 1866 to Miss Genevieve Almira Sprigg, of St. Louis, Mo., who, with his daughter and two grandsons, survived him.

In appearance, Gen. Ludlow was an ideal type of soldier, tall, erect, and graceful, with strong clean cut features. Possessed of a genial manner and a lively sense of humor, his conversation was always interesting and witty. He was a most convincing speaker and writer and his reports were always clear, concise, and logical, possessing an interest rarely associated with public documents. Allotted many and varied duties in his thirty-seven years of active service in the Army, his work was always marked by increased endeavor and results, and as he was ready to praise or censure when such was due, his assistants, civil and military, gave him a devotion and loyalty such as few men are able to inspire.

In character, his strongest traits were his uprightness and hatred of deceit, his devotion to duty, pureness of mind, hospitality, and charity. Gen. Ludlow was deeply religious in his later years and carried his standards into his every day life. His ability, energy, and high aims were continually shown in the furtherance of his plans for increasing the efficiency of the service he so loved and honored, which plans ended for him in their preparation. In all things, he was a man and a gentleman.

The stone covering his burial place contains the following inscription which fittingly describes the soldier and the man:

"A soldier who fought the warfare of life with the same courage and bravery he displayed on the field of battle. Fearless and unswerving in what he believed to be right. Brilliant and versatile, as Engineer, Governor and Commander of troops, he achieved notable success. His life was illumined with bright deeds and with a generous humanity that lifted or shared the burden of others. In the supreme hour of trial his splendid courage was unshaken and he died in the fullest belief of the Life Eternal."

He was a Companion of the Military Order of the Loyal Legion of the United States and a member of various other associations.

Gen. Ludlow was elected a Member of the American Society of Civil Engineers on July 5th, 1882, and served as a Director in 1890.

WILLARD ATHERTON NICHOLS, M. Am. Soc. C. E.*

DIED AUGUST 25TH, 1921.

Willard Atherton Nichols, the son of Dr. George Henry and Sarah (Atherton) Nichols, was born at Standish, Me., on August 22d, 1844. His ancestors were distinguished Colonial and Revolutionary New Englanders. After preparatory work at the Boston Latin and English High Schools, he was graduated from Harvard University (Lawrence Scientific School) in 1865 as Civil Engineer, with the degree of B. S.

His first work was, for a short time, as Assistant Engineer of the Bureau of Sewers of the Croton Aqueduct Department of New York City. He left that position to build the Sullivan County and Erie Railroad in Pennsylvania. When that railroad was completed, Mr. Nichols went to Maine and, as Chief Engineer, took charge of the construction of the railroad then known as the European and North American Railroad, from Bangor, Me., into New Brunswick, which railroad has since been absorbed and consolidated in the Boston and Maine and other roads.

In June, 1876, he was offered and accepted the position of First Assistant Engineer in the Department of Docks of New York City, and he fulfilled the duties of that position efficiently and with distinguished ability until his health failed. In 1890, the Department granted him a leave of absence in order that he might recover his health and resume his work.

Mr. Nichols however did not return to this work, and having been advised or rather ordered to seek a milder climate, he went to Redlands, in Southern California, where he bought land and planted an orange grove which was successful. He also practiced his profession there, as his health and strength permitted, and gave much advice from his knowledge and experience to the people in the vicinity by whom he was very highly respected and very much liked.

Mr. Nichols was a strong and able man and a good citizen, and one of his noticeable qualities was the evenness of his temper under all circumstances.

He did not marry, and is survived by a twin sister, Mrs. Elizabeth K. Hills, of Marblehead, Mass., and two nephews, Henry Atherton Nichols, a banker of Cambridge, Mass., and John Gilman Nichols, a New England manufacturer.

Mr. Nichols was a member of the Society of Colonial Wars, of the Sons of the Revolution, and of the Massachusetts Society of Mayflower Descendants. He was a member of the Unitarian Church of Redlands, Calif., a Trustee of the Smiley Public Library, and a member of other local clubs.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

* Memoir prepared by George S. Greene, Jr., M. Am. Soc. C. E., and Charles H. Myers, Esq.

PHILIP A MORLEY PARKER, M. Am. Soc. C. E.*

DIED AUGUST 4TH, 1920.

Philip A Morley Parker, the younger son of Captain Philip A Morley Parker, R. N., was born at Greenwich, Kent, England, on May 31st, 1872. He was educated at Cheltenham College and his great mathematical ability enabled him to reach the top of the mathematical side before he left. Choosing the Army as a profession, he passed into the Royal Military Academy, Woolwich, but was rejected owing to a weak heart by which he was handicapped throughout his life. In 1889, Mr. Parker entered the Central Technical College (now The City and Guilds of London Engineering College of the Imperial College of Science and Technology) and studied Civil and Mechanical Engineering under Professor Unwin, but on the appointment of his father to the Australian Squadron, he left England before he had completed the course. He continued his engineering studies at Melbourne University and took the Degree of Bachelor of Civil Engineering (B. C. E.) in 1894. On his return to England, he entered Cambridge University, took the Mathematical Tripos in 1897, was 12th Wrangler (notwithstanding illness during the examination), and Senior Scholar of St. John's College.

In May, 1898, Mr. Parker entered the office of Messrs. Hunter and Middleton, Civil Engineers, London, England, and at the end of his pupilage acted as Assistant Resident Engineer on the construction of the Staines Reservoirs—an extensive part of the London Water Supply.

After three years of independent consulting practice in London, he entered the Indian Irrigation Service in 1905, and during the following five years rendered valuable services with respect to design, original investigation, and in an executive capacity, as Assistant Engineer on the project for the Upper Swat Canal, Assistant in charge of the Lower Jhelum Canal Irrigation Area, and in charge of the Second Division of the Upper Chenab Canal. He also served as Executive Engineer in charge of the Upper Jhelum Canal and as Officer in charge of general hydraulic experiments, in pumping on the Upper Bari Doab Canal, and on weir and sluice discharges, etc.

Resigning from the Punjab Irrigation in 1910, Mr. Parker began his book, "The Control of Water", and in order to complete his study of irrigation and general hydraulic practice, he visited the United States, Japan, Ceylon, Egypt, and Italy. While occupied on his book and up to the beginning of the World War, he continued his consulting practice in London and, during that period, carried out some irrigation projects in Jamaica.

In the early stages of the World War, when the fulfillment of urgent requirements largely depended on the initiative of individuals, Mr. Parker did extremely valuable work. With several friends, he gave lectures to a large number of Army Service Corps recruits on water supply in the field and practical demonstrations of the application of the best safeguards to neutralize

* Memoir prepared by J. S. Wilson, Esq., London, England.

harmful qualities in water. He also printed and distributed to officers and men, at his own expense, a vocabulary relating to the subject in English, French, and German, with which was incorporated some concise instructions. Early in 1915, before any Government departmental system had been set up to deal with water questions, Mr. Parker undertook to provide a water supply for a large plant which was being transformed into an explosives factory by Lord Moulton's newly formed committee. The plan which he prepared and carried out, included the laying of an 18-in. water main, 8 miles long, from the Halkyn Mountains to Queensferry on the Dee. The active co-operation of a contractor and the use made by Mr. Parker in his executive capacity of the emergency powers to overcome obstacles, enabled him to complete the work in a phenomenally short time. In laying the pipe, a maximum rate of 1 400 ft. per day was attained.

The winters proved trying to his health and he left London for the last time in May, 1916, to take the appointment of Chief on the Constructional Staff on the Sydney Underground City Railway. Work on this railway was discontinued in 1917, owing to a change of policy by the Government. From Sydney, Mr. Parker went to the Federated Malay States, and there irrigation projects in connection with land development proved to be his last work, for while thus engaged he died of heart failure at Kuala Lumpur, on August 4th, 1920.

By his death the Engineering Profession has lost one of its younger and more highly trained exponents. Mr. Parker's personality was an attractive one and, coupled with his great ability, gave him a large circle of friends; generally, however, he was best known as the auther of "The Control of Water" published in 1913. This work covers almost the whole science of Hydraulic Engineering, and Mr. Parker, in addition to a profound knowledge of the underlying principles, had the advantage of a wide practical experience, a combination which places the book in a unique position among manuals on "Hydraulics".

Throughout his career, Mr. Parker was a keen student of engineering, particularly of the branch he had made his own. He had carefully followed all original investigations and experiments reported from all parts of the world and, in his own work, whenever there was an opportunity of making an investigation, he made full use of his ability with the object of converting the results of his experience into rules or principles of general application, and in doing this he carefully considered any other work of importance bearing on the matter. In "The Control of Water", every subject throughout the wide range covered is reviewed and examined in a masterly manner and with a completeness of reference and cross-reference which makes the book not only the best in the language on the subject, but "a monumental contribution to applied hydraulics", as it was described in a review by an eminent American authority. Mr. Parker had other books in preparation at the time of his death.

Mr. Parker was elected an Associate Member of the American Society of Civil Engineers on March 6th, 1907, and a Member on January 6th, 1915. He was also a Member of the Institution of Civil Engineers.

EDMUND TAYLOR PERKINS, M. Am. Soc. C. E.*

DIED MAY 21ST, 1921.

Edmund Taylor Perkins was born at Scottsville, Va., on September 8th, 1864. His father was Edmund Taylor Perkins, of Virginia, and his mother, Mary Addison, of Maryland. He was graduated from Union College, Schenectady, N. Y., in 1885, with degrees of C. E. and A. B., and received the degree of M. A., in 1888.

Mr. Perkins served with the U. S. Geological Survey from 1885 to 1902. Later, he was with the U. S. Reclamation Service (1902-1909) in charge of stream gauging and the determination of run-off factors in Colorado, and of the preliminary surveys and plans for the Yuma Irrigation Project in Arizona; he also served as General Inspector of the Chicago Transportation and Contracting Office.

In 1910, he established the present office of the Edmund T. Perkins Engineering Company, in Chicago, Ill. As a Consulting Engineer, Mr. Perkins specialized in land drainage, reclamation, and flood control. He was a colleague of the late Isham Randolph, M. Am. Soc. C. E., and with him served as a member of the Everglades Engineering Commission of the State of Florida. Mr. Perkins was Engineer for the Green Bay Levee and Drainage District No. 2 of Lee County, Iowa; the South Quincy and Valley City Drainage Districts of Illinois; the Marion County Drainage District of Missouri; and also acted as Consulting Engineer for several large drainage districts of Arkansas.

He had just returned from a trip to South America where he had been engaged on an investigation and report of the resources of Colombia with a view to extensive development there, when he suffered a sudden attack of heart failure from which he died at his home in Chicago, Ill., on May 21st, 1921. He was buried at his boyhood home in Louisville, Ky. Mr. Perkins is survived by his wife, Louise Scribner Perkins, formerly of Washington, D. C., and two sisters.

He was an active and loyal worker in the engineering societies of which he was a member. He was President of the National Drainage Congress which he organized on December 8th, 1920; and a member of the Board of Directors and Past-President of the American Association of Engineers. He was also a member of the Western Society of Engineers, the Illinois Society of Engineers, and the Engineers' Clubs of New York City and Chicago.

Mr. Perkins was also active in the work of the Mississippi Valley Association and the Chicago Association of Commerce, and was a member of the University Club, Iroquois Club, and Glen View Club of Chicago, and the Chevy Chase Club of Maryland.

He was a man of genial disposition and had won many friends. He believed that the engineer should take his part in the duties of citizenship and public affairs.

Mr. Perkins was elected a Member of the American Society of Civil Engineers on December 3d, 1902.

* Memoir prepared by L. K. Sherman, M. Am. Soc. C. E.

HIRAM PHILLIPS, M. Am. Soc. C. E.*

DIED DECEMBER 22D, 1921.

Hiram Phillips was born in Boone County, Missouri, on November 9th, 1859, and received his early education in the public schools of that county. Later, he attended the Missouri State University, and was graduated in the Engineering Class of 1880.

His first employment was with the Mississippi River Commission on surveys and construction on the Lower Mississippi. After more than three years of this work, he resigned in 1884 to take a Post-Graduate Course at the University of Missouri in Mining Engineering and Assaying.

Mr. Phillips then secured a commission as District Mineral Surveyor of Colorado, opening an office at Alma, Colo., where, in partnership with Mr. W. W. Powliss, he followed this branch of engineering for about four years. He then returned to Columbia, Mo., where he accepted an appointment as Assistant Professor of Engineering in the University of Missouri. Being an intensely practical man, Mr. Phillips found his eight years in the field and office of inestimable value to him as a teacher. Many practicing engineers to-day can attribute some measure of their successful practice to the common-sense way in which engineering problems were placed before them by Hiram Phillips during their college days.

In 1891, Mr. Phillips moved to St. Louis, where he maintained an office during the remainder of his life. He engaged in general engineering work, but devoted the greater part of his time to the design and construction of water-works and sewerage systems.

In 1901, he was elected President of the Board of Public Improvements of the City of St. Louis for a term of four years. Under the charter of the city at that time, the President of the Board of Public Improvements exercised supervision over the five Departments, the heads of which composed the Board under which all the public work done in the city was designed and executed, namely, the Water, Sewer, Harbor, and Park Departments, Commissioners for the first three being required to be engineers, as was also the President.

During Mr. Phillips' term of office, in addition to the construction being carried on at all times in a city of the size of St. Louis, which required more or less attention from his office, the World's Fair was built, held, and cleared away, all of which entailed a great amount of detailed work on the President and his office, and brought up many difficult questions for settlement. Mr. Phillips' fairness and clean-cut sense of justice brought him through this trying time with credit, not only for himself, but for all his associates on the Board.

In 1905, he resumed his private practice, with St. Louis as his headquarters. During his professional life he served about sixty-nine cities and towns as Consulting, Designing, or Constructing Engineer. He was appointed

* Memoir prepared by the following Committee of the St. Louis Section: S. Bent Russell, John T. Garrett, and Edward E. Wall, Members, Am. Soc. C. E.

on many boards and commissions and reported on various projects for many places, among which may be mentioned Denver, Colo., Oklahoma, Okla., Palm Beach, Fla., Kansas City, Mo., Colorado Springs, Colo., and Santa Fe, N. Mex.

Some of the places for which Mr. Phillips designed and built public works are Florence, Colo., Brookfield, Mo., Columbia, Mo., Raton, N. Mex., Mariana, Blytheville, and Walnut Ridge, Ark., St. Charles, Mo., Centralia, Ill., Grand Junction, Colo., and Phœnix, Ariz. The infiltration galleries on the Verde River and the 30-mile pipe line for the new water supply for Phœnix had just been completed, the water being turned on about December 15th, 1921, and, on December 22d, 1921, Mr. Phillips was making a final inspection of the work, which he had fathered from its beginning in 1912, and for which he had acted as Consulting Engineer to its completion, when he met with the fatal accident which caused his death. The automobile in which he was riding overturned and crushed him.

Hiram Phillips was an engineer who applied common sense to all problems to an uncommon degree. He was outspoken in his views and tenacious of his opinions, intensely loyal to his friends, and gifted with the faculty of making and remembering acquaintances. He was a man of great energy, with a restless mind, ever devising means of getting work under way, and continually seeking new and better methods of construction.

In 1885, he was married to Miss Nellie Horine who, with two children, Paul and Elizabeth, survives him.

Mr. Phillips was a Mason, an Elk, a member of the St. Louis Engineers' Club, of the American Water Works Association, of the Noonday Club of St. Louis, and of other technical and social societies and organizations.

Mr. Phillips was elected an Associate Member of the American Society of Civil Engineers on January 3d, 1894, and a Member on November 3d, 1897.

JOHN CHARLES QUINTUS, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1921.

John Charles Quintus, the son of Jacob and Catherine Quintus, was born in Sheboygan, Wis., on October 8th, 1856. His parents were Hollanders, who came to the United States in 1847, his maternal ancestors having been French Huguenots who fled to Holland to escape religious persecution.

In 1858, the family moved to Grand Rapids, Mich., where Mr. Quintus received his elementary education. He entered the University of Michigan in 1875, and was graduated in 1879, with the degree of Mining Engineer.

He practiced mining engineering for about one year after leaving college, and then entered the Government service with the Mississippi River Commission at St. Louis, Mo., and for about six years was employed principally on triangulation work on the Mississippi and Missouri Rivers. In 1886, he was transferred to the U. S. Lake Survey, and was stationed at Grand Rapids, Mich. In 1887, he began work in the Buffalo District, Engineer Department at Large, at Erie, Pa., where he remained for eight years, in local charge of harbor improvement work, consisting of dredging, pier and jetty construction, and shore protection work.

In 1895, he was transferred to Buffalo, N. Y., and shortly after the transfer became Principal Assistant Engineer of the Buffalo District, in which position he served until February, 1915. On February 26th, 1915, Mr. Quintus was promoted, and appointed Assistant Engineer in the office of the Division Engineer, Lakes Division, at Buffalo, N. Y., aiding in the supervisory charge of all the harbors and connecting channels of the Great Lakes and tributaries, from Duluth, Minn., to Ogdensburg, N. Y., in which position he served until his death.

On December 24th, 1886, Mr. Quintus was married to Miss Nellie Nellis, of Mount Clemens, Mich., who, with one daughter, Katrina, survives him.

Mr. Quintus designed the rubble mound breakwater for the extension of the Buffalo Breakwater System to Stony Point. This structure, with its closely paved top of large stone and long sloping lake face, is and will remain a monumental structure. He also designed the Black Rock Ship Lock and the steel sheet-pile coffer-dam that was used in the construction of the lock. This is believed to be the first time that this type of square, clay-filled, connected pockets of steel sheet-piling was ever used for a coffer-dam. A modified form of this type of coffer-dam was used later for raising the U. S. S. *Maine,* in Havana Harbor, and has subsequently been used quite extensively where limited room makes a costly type of dam necessary.

Mr. Quintus was a member of the Chi Psi Fraternity and the Westminster Club, and, at one time, he was also a member of the University Club of Buffalo, but being a home-loving man, he did not maintain his membership in this club.

He was a quiet, kindly man who, although decided in his opinions and exact in his methods, was never harsh in his treatment of his subordinates. He was remarkably well versed in the laws and regulations pertaining to

* Memoir prepared by Lynn L. Davis, M. Am. Soc. C. E.

the Engineer Department at Large, both in letter and spirit, as well as in the design and construction of work for the improvement of the harbors and connecting channels of the Great Lakes; and this knowledge was of great value to his superiors, to whom he was thoroughly loyal, faithfully carrying out their ideas, even although at times they were at variance with his own, and he was highly esteemed by all of them.

His unswerving fidelity to the Department which he served for more than forty years can be said to be his outstanding characteristic. In the words of one of the Senior Colonels of the Corps of Engineers, with whom Mr. Quintus had served continuously for the past eleven years, "He was a most valuable public servant."

Mr. Quintus was elected a Member of the American Society of Civil Engineers on January 2d, 1889.

GEORGE STAPLES RICE, M. Am. Soc. C. E.*

DIED DECEMBER 7TH, 1920.

George Staples Rice was born in Boston, Mass., on February 28th, 1849. He came of sterling New England stock, his ancestors having borne their part in constructing the history of New England in the old Colonial days. His father was Reuben Rice, a direct descendant of Edmund Rice, who emigrated from England to the United States in 1638, and was one of the early settlers of Marlborough, Mass. His mother, Harriet Tyler (Ketelle) Rice, also came of old English ancestry.

His early education was received in the public schools of Boston. Afterward, he entered Harvard University and was graduated in 1870, with the degree of S. B.

Even before his course at Harvard was finished, Mr. Rice began his public work. At that time, municipal engineering in the United States was just beginning. Previous to his last year at the University, Mr. Rice spent the summer in the service of the Engineering Department of the Boston Water-Works, assisting in the construction of the Chestnut Hill Reservoir. After his graduation from Harvard, he became Assistant Engineer of the Lowell, Mass., Water-Works, and, later, Assistant Division Engineer of the Boston Water-Works. From 1877 to 1880, he filled the position of Assistant Engineer and Principal Assistant Engineer in charge of the Boston Main Drainage Works, which was one of the most important sanitary engineering projects ever undertaken by the City of Boston.

At this time, the late James B. Francis and Alphonse Fteley, Past-Presidents, Am. Soc. C. E., and Joseph P. Davis, M. Am. Soc. C. E., all pioneer hydraulic engineers, were practising their profession in Boston, Mr. Davis having been City Engineer of Boston during this time. The late Hiram F. Mills, Hon. M. Am. Soc. C. E., also belonged to this group. Daily contact and association with such men was an inspiration to Mr. Rice, then a young engineer, and greatly influenced his whole professional career.

In 1880, however, the lure of the West became too strong for him, and he went to Arizona and Colorado where he was engaged in mining operations. This work occupied him for seven years, when the call came for him to return East.

His previous experience on the public works of Boston brought him the appointment, in 1887, of Deputy Chief Engineer of the Aqueduct Commission of New York City. Thus, he began his work for the city to which he was destined to give the greater part of his life's service.

In 1891, Mr. Rice returned to his native city to become Chief Engineer of the Boston Transit Commission. He made a thorough investigation into the transit conditions of the city, and prepared a comprehensive report which was an important factor in the future development of the transit facilities of Boston. He remained in Boston until 1900. During this period (1891-1900), he was also engaged in the private practice of engineering with

* Memoir prepared by D. L. Turner, M. Am. Soc. C. E.

the late George E. Evans, M. Am. Soc. C. E., under the firm name of Rice and Evans. This firm designed and constructed the water-works of New Bedford, Mass.

While practising his profession in Boston, Mr. Rice received a call to aid his Alma Mater in developing its School of Engineering from which he had been graduated twenty years before, and to this call he gladly and loyally responded. For eight years (1892-1900), Mr. Rice served Harvard University as Instructor in Sanitary Engineering.

Meanwhile, his work for the Boston Transit Commission had attracted attention in New York City where improved transit facilities had become a vital necessity. William Barclay Parsons, M. Am. Soc. C. E., Chief Engineer of the Board of Rapid Transit Railroad Commissioners, appointed him Deputy Chief Engineer of the Commission, and he served with Mr. Parsons during the construction of the first subway for New York City. When Mr. Parsons resigned on December 31st, 1904, Mr. Rice succeeded him, first as Acting Chief, and then as Chief Engineer, of the Commission.

In 1907, shortly after the Public Service Commission succeeded to the work of the Rapid Transit Board, Mr. Rice became Engineer of Subway Construction, but he resigned this position in 1910 to engage in private practice. Later, in 1914, when the work of constructing the Dual Subway System for New York was begun, he returned to the service of the City and became a Division Engineer in charge of the construction of a large part of the work, which position he held until his death on December 7th, 1920.

Mr. Rice served his native city, Boston, and his adopted city, New York, in a highly honorable and unselfish manner throughout thirty years of his professional career; in other words, he devoted nearly two-thirds of his professional life to public service.

He was married in Yonkers, N. Y., on October 10th, 1889, to Rose Breuchaud, who, with one son, Albert F. Rice, survives him.

His long and active life brought him many friends, to all of whom he was greatly endeared because of his lovable and unselfish character.

At his death, the Faculty of Harvard University adopted the following minute:

"George Staples Rice, S. B. 1870, was a most loyal and useful friend of Harvard University. He rendered distinguished service to the University by a highly honorable and unselfish career as the director of great public works vitally affecting the welfare and safety of millions of people, included in which are some of the most important engineering enterprises of his time, such as the present water supply, drainage, and rapid transit systems of Boston, the New Croton Aqueduct, and the great subway system of New York.

"Mr. Rice served the University devotedly as Instructor in Sanitary Engineering for a period of eight years (1892-1900), after the close of his work as Deputy Chief Engineer for the New Croton Aqueduct—in the full tide of his life of great achievement.

"Mr. Rice's official relations with the University terminated when he was called to New York to become Deputy Chief Engineer, and soon after, Chief Engineer of the New York subway development; but this did not close his direct co-operation with Harvard work in Engineering. Through his foresight, his friendly advice, and encouragement, in the early days of the New York Sub-

way, he made his own opportunities count as opportunities also for scores of younger men from Harvard. Thus, he, perhaps above all others of our graduates for the past fifty years, opened the way by which the University, through these young men, is steadily extending its influence for the convenience and safety of the public, and by which the young men themselves are securing the lasting satisfaction which comes from doing great and worthy tasks well.

"The Faculty of Engineering accordingly and with deep gratitude records its appreciation of the inestimable value of Mr. Rice's energetic and unceasing loyalty to Harvard University."

He was a member of the Boston Society of Civil Engineers, the American Institute of Mining and Metallurgical Engineers, the New England Water-Works Association, and the Society of Colonial Wars. His clubs were the University, Harvard, and Arkwright of New York City, and the Union and St. Botolph of Boston, Mass.

Mr. Rice was elected a Member of the American Society of Civil Engineers on February 1st, 1882.

WILLIAM HARPER ROBINSON, M. Am. Soc. C. E.*

DIED DECEMBER 29TH, 1920.

William Harper Robinson, the son of Judge Robert Robinson and Luisa Harper Robinson, was born on February 10th, 1868, at Sacramento, Cal. He was educated in the public schools of San Francisco, Cal., and after having been graduated from the Boys' High School of that city, attended Heald's Business College, the Mechanics' Institute of San Francisco, and the Van der Naillen School of Engineering.

In 1887, he entered the employ of the Southern Pacific Railroad Company, for which Company he worked continuously, as Draftsman, Leveler, Transitman, and Assistant Engineer, for a period of ten years. During that time, Mr. Robinson was Draftsman on the following Southern Pacific surveys: From Santa Ana to Oceanside, Burbank to Chatsworth Park, Long Beach Junction to Long Beach, San Pedro to Point Fermin, Ventura to Nordhoff, Montalvo to Simi Pass, Oakdale to Merced, and on double-track construction from Oakland to Port Costa, all in California. During 1889, he served as Draftsman in the General Office of the Company in San Francisco.

In 1890, Mr. Robinson was employed as Leveler on the following Southern Pacific surveys: From Riverside to San Bernardino, Colton to Redlands, and Avon to Walnut Creek. During the same year, he was Transitman on the line from Santa Monica to Soldiers' Home, and on various main-line changes in California, Arizona, and New Mexico.

In 1891, he was engaged as Transitman and Topographer for the Ferrocaril Occidental de Guatemala, on the line between San Felipe and Quezaltenango.

The year 1892 found Mr. Robinson again with the Southern Pacific Railroad Company as Draftsman in the General Office in San Francisco. In 1893, he was engaged as Transitman on surveys for the Company from Gaviota Landing to Lompoc Landing and from San Luis Obispo to Lompoc Landing, and, in 1894, he was employed as Assistant Engineer on line changes on the Park and Ocean Railroad, on relocation from San Bruno to San Francisco, and on line changes between San Miguel and Santa Margarita. In 1895, he was again in the General Office in San Francisco. In 1896, he served as Assistant Engineer on the construction of standard road between Riverside and Colton and on water development work on the Hope Ranch in Santa Barbara County, California.

In 1897 and 1898, Mr. Robinson was engaged in the private practice of his profession, largely on surveys, land irrigation, and water development. In 1899-1900, he was employed by the Redlands Electric Light and Power Company on the location and construction of the power plants in Mill Creek Canyon, San Bernardino County, California.

In 1900, he went to the Philippine Islands where he was engaged as Leveler and Surveyor on the Benguet Road, and, in 1901, he became Assistant to the Superintending Engineer of the Army Transport Service at Manila.

* Memoir prepared by L. Fred Patstone, M. Am. Soc. C. E.

In 1902, he was appointed as Supervisor of the Province of Misamis and, in 1903, was transferred to the post of Assistant Engineer to the Philippine Commission. Soon after this, he was transferred to the Coast Guard Service as Principal Assistant Engineer in the Division of Lighthouse Construction. Mr. Robinson resigned this position, in 1905, to accept that of Superintendent of Buildings for the Philippine Railroad Company, which position he held for three years, during which time he designed and built all the depots and terminals of that road.

In the fall of 1908, he again entered the Philippine Government Service as Division Road Engineer in the Bureau of Public Works, and from this time, his promotion in that service was rapid. He passed through the positions of Division Engineer, Chief Division Engineer, Principal Assistant Engineer, and Acting Assistant Director, until on June 6th, 1910, he was appointed City Engineer of Manila and Head of the Department of Engineering and Public Works, which position he held until August 1st, 1916.

During this time, Mr. Robinson was a big factor in developing the City Plan for the future improvement of Manila and in establishing and improving many of its public works. As *ex-officio* member of the Municipal Board of Manila, he was able to give engineering advice and guidance in many of the great problems incident to the growth and well-being of the city.

Owing to ill-health, Mr. Robinson was forced to resign in August, 1916, and return to the United States. He purchased a ranch in Marin County, California, where he continued to reside until his death which occurred on December 29th, 1920. He is survived by his wife, Emma Upchurch Robinson, to whom he was married in San Diego, Cal., on March 4th, 1896.

After Mr. Robinson's return to the United States, ill-health prevented him from engaging in active work, although he retained a keen interest in his chosen profession. At the time of his death, he was in charge of the street and paving improvements in Dixon, Cal.

Mr. Robinson was an exceptionally fine man with whom to work, and all engineers who came in contact with him appreciated his broad-mindedness and his spirit of co-operation. He was the type of man who is progressive and is not afraid to shoulder responsibility. Throughout the Philippines, and especially in Manila, there are many public works which stand as monuments to his memory and as mute testimonials to a long and successful career in the engineering work of the United States Government in the Philippines.

Mr. Robinson was elected a Member of the American Society of Civil Engineers on March 1st, 1910.

JOHN RICHARD SAVAGE, M. Am. Soc. C. E.*

DIED FEBRUARY 25TH, 1922.

John Richard Savage was born in Philadelphia, Pa., on April 17th, 1869. He was graduated from the University of Pennsylvania in 1889, receiving degrees of Bachelor of Science and Civil Engineer.

In May, 1889, Mr. Savage entered the employ of the Pennsylvania Railroad Company as Rodman and Levelman in the Chief Engineer's Department, where he served until October, 1890, resigning to accept a position under Capt. T. W. Symons, Corps of Engineers, U. S. A., with headquarters at Portland, Ore.

Mr. Savage continued in the service of the Government, as Assistant Engineer, until May, 1895, during which period he had charge of reconnaissances, hydrographic surveys, and harbor improvements in the Puget Sound country. In May, 1895, he was appointed Engineer of the Seattle and Lake Washington Company, a corporation the plans of which, in connection with the reclamation of tidal lands adjoining Seattle, Wash., and the cutting of a canal from Puget Sound to Lake Washington, contemplated the present development.

In August, 1897, Mr. Savage again entered the employ of the Pennsylvania Railroad Company, under the Engineer of Branch Lines, and was engaged on the construction of the Norfolk and Portsmouth Belt Line Railroad, of piers at Norfolk, Va., and of a part of the Southwest Pennsylvania Railroad extension in Fayette County, Pennsylvania. In 1899, at the close of the Spanish-American War, the Federal Government appealed to the Pennsylvania Railroad Company for an engineer to complete the construction of railroads and docks in the environs of Havana, Cuba, and Mr. Savage was selected for this task.

From October, 1900, to July, 1901, he was engaged in making surveys for a proposed railroad through Lebanon Valley from Cornwall to Highspire, Pa., a new outlet for the Cornwall ore banks. In August, 1901, he entered the employ of the Lackawanna Iron and Steel Company, later known as the Lackawanna Steel Company, and continued with that firm during the construction of the new plant at Buffalo, N. Y., holding the positions of Superintendent, Assistant to the Vice-President, and Assistant General Superintendent.

On April 1st, 1904, Mr. Savage was appointed Chief Engineer of the Long Island Railroad and from that date until October, 1917, he was in direct charge of the large and diversified engineering and construction projects on Long Island, the most important being the Jamaica Improvement, which was an extended elimination of grade crossings, and the construction of the most intensive passenger transfer facilities in the United States, all accomplished during train operation, the grade-crossing elimination and revision of the main line, connecting this Transfer Station with the Sunnyside Yard of the Pennsylvania Terminal, and the changes in grade through Flushing. On

* Memoir prepared by L. V. Morris, M. Am. Soc. C. E.

October 1st, 1917, he was appointed General Manager of the Long Island Railroad, succeeding Col. James A. McCrea who went to France with the American Expeditionary Forces.

During the period of Federal control, Mr. Savage was General Manager of the Long Island Railroad, under Mr. Ralph Peters, Federal Manager. With the re-organization, on March 1st, 1920, at the end of Federal control, the position of General Manager was abolished, and Mr. Savage was made General Superintendent with practically the same duties.

Mr. Savage was married on June 7th, 1904, to Miss Elspeth Mona Murray who, with three children, survives him.

Mr. Savage's military and gentlemanly personality appealed to all who knew him. His outstanding characteristic was that of minute attention and interest in the details of design, construction, operation, and accounting. He was endowed with the faculty for thorough investigation and analysis and a clear, concise, and careful expression of conclusions. His untiring devotion to his duties, his ever-listening ear, his personal interest as adviser to all members of his staff, make his loss the more pronounced professionally as an engineer and humanly as a man.

To few men in railroad life was it given to hold the respect and affection of his fellow officials, as well as that of the employees generally, to the degree enjoyed by Mr. Savage. Blessed with a kindly disposition, coupled with a broad and philosophical view of life, he was equipped to handle big problems successfully and to win the ardent co-operation of the men with whom he worked so loyally.

Mr. Savage was a member of the following clubs and technical associations: Garden City Club and Cherry Valley Club, Garden City, N. Y.; Huddersfield Fish and Game Club, Canada; University of Pennsylvania Club of New York; Psi Upsilon Fraternity; Engineering Alumni Society of the University of Pennsylvania; General Managers' Association of New York; and the American Railway Engineering Association.

Mr. Savage was elected a Member of the American Society of Civil Engineers on June 7th, 1905.

WILLIAM HENRY SEARLES, M. Am. Soc. C. E.*

DIED APRIL 23D, 1921.

William Henry Searles, the son of Ashbury M. and Rachel Mitchell Searles, was born in Cincinnati, Ohio, on June 4th, 1837.

He attended private schools for preparatory work for Wesleyan University, at which he was a student in 1856 and 1857. Later, he entered Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in 1860.

In 1861, Mr. Searles served as Assistant Engineer of the Marietta and Cincinnati Railroad Company, and as Military Engineer on the staff of Gen. Buell and under Gen. Rosecrans, on the defenses of Cincinnati.

His interest in technical and scientific work led to his service as Professor of Geodesy and Topography at his Alma Mater in 1862 and 1864.

In 1865, he became Chief Engineer of the Mineral Range Railroad of Michigan, now a part of the Duluth, South Shore and Atlantic Railroad, and, in 1866, Assistant Engineer of the Allegheny River Bridge, for the Pittsburgh, Fort Wayne and Chicago Railroad Company. In 1867, Mr. Searles was employed as Principal Assistant Engineer of the Allegheny Valley Railroad Company.

During 1868-69, he was engaged as a manufacturer of petroleum products at Cleveland, Ohio, but in 1870-71, he again engaged in the practice of his profession as Chief Engineer of the Indiana North and South Railroad Company, at Indianapolis, Ind., and, in 1872-73, as Chief Engineer (7th Corps) of the New York and West Shore Railroad, now a part of the New York Central System.

From 1876 to 1878, Mr. Searles acted as Consulting Engineer on the New York State Canals, following which, he entered private practice as a Consulting Engineer, in Cleveland, Ohio.

After retiring from active work, Mr. Searles took up his residence in Elyria, near Cleveland, Ohio, where he was actively engaged in civic and church affairs. For a time he was Superintendent of the Sunday School of the First Congregational Church which he also served as Clerk for thirteen years and as Clerk Emeritus until his death.

He was well known as the author of Searles' "Field Engineering" which was published in 1880, and of the "Railroad Spiral", which was issued in 1882.

From 1888 to 1890, Mr. Searles was a member of the Board of Managers of the Cleveland Engineering Society, in the work of which he took a deep interest. He was President of this Society in 1890 and 1891, and, later, in 1897, served as its Secretary. In 1907, in recognition of his engineering and scientific attainments, he was elected an Honorary Member.

He was married in 1870 to Mary L. Doolittle who survives him, the fiftieth anniversary of their wedding having been observed by Mr. and Mrs.

* Memoir prepared by W. P. Brown and James Ritchie, Members, Am. Soc. C. E., and the late Frank C. Osborn, M. Am. Soc. C. E., a Committee of the Cleveland Section of the American Society of Civil Engineers.

Searles and their many friends at Elyria about a year before his death, which occurred on April 23d, 1921.

Mr. Searles possessed the faculty of a thorough and correct analysis, as well as a clear and concise expression of his conclusions, and, in addition to his attainments in the Engineering Profession, he will be remembered as a Christian gentleman who always strived for high ideals in his professional and private affairs.

Mr. Searles was elected a Member of the American Society of Civil Engineers on July 2d, 1873.

MAX EVERHART SMITH, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1921.

Max Everhart Smith was born in Berlin, Germany, on October 18th, 1848, of German parents, and was named Max Eberhardt Schmidt, the English equivalent having been assumed in 1918, by permission of the Court. He entered the Preparatory Military Academy at Potsdam in 1859, and a higher Military Academy at Berlin in 1863. In 1867, at the age of 19, he was commissioned a Second Lieutenant of the Artillery Guards, the regiment in which his father had also served as an officer. He was graduated from the Academy of Artillery and Engineer Officers in September, 1869, and was honorably retired in March, 1870.

On the outbreak of the Franco-Prussian War in 1870, Mr. Smith was recommissioned, and was in active military service until the fall of Belfort, in April, 1871.

In May, 1871, he came to America, and, from 1871 to 1875, served as Engineer on the United States Government surveys west of the 100th Meridian, under Lieut. George M. Wheeler, Corps of Engineers, U. S. A.; in the Coast and Geodetic Survey, under the late Professor J. E. Hilgard, M. Am. Soc. C. E.; and on Government work in Minnesota and Dakota, under the late Maj. F. U. Farquhar, Corps of Engineers, U. S. A., M. Am. Soc. C. E. Mr. Smith became a naturalized American citizen in 1874.

In 1875, he was appointed by the late Capt. James B. Eads, F. Am. Soc. C. E., as Chief Assistant Engineer on the construction of the South Pass Jetties at the mouth of the Mississippi River, and continued on this work until 1879.

From 1879 to 1881, Mr. Smith was employed as Assistant Engineer on the Government works for the improvement of the Mississippi River at Memphis, Tenn., under Maj. Benyuard, Corps of Engineers, U. S. A., and at St. Louis, Mo., under Maj. O. H. Ernst, Corps of Engineers, U. S. A.

In 1881, he was appointed Engineer in charge of location of the Mexican Central Railroad, on the Tampico-San Luis Potosi Division, and became Chief Engineer of the road in 1884, serving until 1889, when he went to Chicago, Ill.

In 1890, Mr. Smith opened an office in Chicago and engaged in a general consulting practice which he continued until 1894. During this period he conceived the idea of the continuous train, or moving platform, for the transportation of passengers, and constructed a typical section which was operated at the Columbian Exposition in 1893, the Berlin Exposition in 1895, and, later, at the Paris Exposition in 1900. On each occasion it proved so successful and attracted so much favorable comment both from engineers and the general public that Mr. Smith devoted the greater part of the remainder of his life to its introduction for public use. In 1904, he became President and Chief Engineer of the Continuous Transit Securities Company.

* Memoir prepared by Gustav Lindenthal, M. Am. Soc. C. E.

His designs and plans underwent various changes and improvements from time to time, to meet varying conditions, but the general scheme remained the same throughout his entire work. In 1906, he was awarded the John Scott Legacy Medal and Premium by the Franklin Institute of Philadelphia, Pa.

As a result of his activities, Mr. Smith had the satisfaction of seeing his project gain increasing favor and recognition among engineers, and several routes were laid out for its introduction into the transit system of New York City. As he left it, the platform was regarded by transit specialists as the correct solution of the problem of economical and rapid transportation of passengers in congested districts.

Mr. Smith had contributed a number of papers and discussions to the *Transactions* of the Society, notably his paper on "The South Pass Jetties,"* which gained for him the award of the Norman Book Prize by the Society in 1879.

In 1875, he was married to Miss Mary Everhart, of American Colonial and Revolutionary descent, a niece of the late George Plumer Smith, of Philadelphia, Pa., and a great-granddaughter of the Hon. George Plumer, and of Col. Alexander Lowrey, of Pennsylvania.

His death on January 24th, 1921, was a grievous shock to all who knew him. His eldest son, Eads Everhart Smith, died two years before his father. His widow and one son, George Plumer Smith, senior member of the firm Smith and Gallatin, survive him.

Personally, Mr. Smith was a man of extreme refinement and of musical taste and ability. He had composed a number of marches, the last having been a military march entitled "Prepared", which was published by Schirmer and dedicated to the Engineer's Training Battalion of New York. He was a genial companion, an engineer of far vision, a man of unusually broad acquaintance, and left many friends.

This memorial of an Engineer who had been in the front rank of progress and professional attainments, of a man of exceptional culture and most sympathetic and warm friendships, is only an inadequate expression of their high esteem and mourning for him by those who knew him long and well.

Mr. Smith was elected a Member of the American Society of Civil Engineers on May 7th, 1879.

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* *Transactions*, Am. Soc. C. E., Vol. VIII (1878), p. 189.

GEORGE DUNCAN SNYDER, M. Am. Soc. C. E.*

DIED OCTOBER 21ST, 1921.

George Duncan Snyder, the son of George Stephen and Anna Wesley (Butler) Snyder, was born in Williamsport, Pa., on September 25th, 1866, and received his early education in the public and private schools of Pennsylvania.

His American ancestors belong solely to Pennsylvania, having come to that State between 1750 and 1790, and were of English, Irish, Scotch, French, and German extraction. They and their descendants were prominent as soldiers, sailors, engineers, ministers, and public officials.

From 1882 to 1886, Mr. Snyder was engaged in the office of his uncle, Antes Snyder, at that time Engineer of Right of Way of the Pennsylvania Railroad, and was employed on re-surveys of right of way, in addition to railroad location and construction, for that Company.

Later, he went West, on the lines of the Union Pacific Railroad, and during 1886-87 was engaged on branch line construction. He returned to the East, however, in the latter part of 1887, and from that time until 1890, was Assistant Supervisor on Maintenance of Way of the Pennsylvania Railroad.

From 1890 to 1893, he transferred his affiliations to the Philadelphia and Reading Railroad, becoming Assistant Engineer on the Williamsport Division. He was engaged in the re-alignment of the railroad of that Company, as well as in the layout of coal storage yards and the location of branches and connecting lines. During this period, Mr. Snyder had full charge of the preliminary and location surveys of the Southern Central Railroad along the west side of the Susquehanna River, from opposite Sunbury to near Harrisburg, Pa.

During 1893, he still further transferred his allegiance to the Erie Railroad, having been engaged in the New York Office of the late C. W. Buchholz, M. Am. Soc. C. E., on studies for and renewals of the structures of that Company.

In 1894, Mr. Snyder was appointed to the office of City Engineer by the authorities of his home town, Williamsport, which office he ably filled for a period of seven years, during which time he was responsible for the construction of the storm and drainage sewer system. During this period of service, he acted as a member of a Board of Engineers, with the late C. W. Raymond, M. Am. Soc. C. E., then Major of Engineers, U. S. A., and, later, Brigadier-General, U. S. A., and the late L. Y. Schermerhorn, M. Am. Soc. C. E., in making a report on the protection of the City of Williamsport from floods of the Susquehanna River.

In 1902, Mr. Snyder joined the staff of Jacobs and Davies, Consulting Engineers, and was engaged to proceed to Burma, India, where, for about a year, he carried out surveys and made a report and estimates for an oil pipe line of about 360 miles for the Burmah Oil Company, extending from the

* Memoir prepared by J. V. Davies, M. Am. Soc. C. E.

refineries at Rangoon up the Valley of the Irrawaddy River to the oil fields at Yenang-Yat. After his return from Burma, he was further associated with the purchase and inspection of material for this pipe line which was constructed on the lines of the surveys made by him.

On the conclusion of this work, the construction of the underground railroad system of the Hudson and Manhattan Railroad was in active progress, and Mr. Snyder was transferred to the staff of the Hudson Companies as Principal Assistant Engineer on that work. With the work of construction of this underground railroad, he maintained connection to the time of the delivery of the railroad to the operating company, and, thereafter, as Deputy Chief Engineer of the Hudson and Manhattan Railroad Company. During this period of about 13 years, he was engaged for the most part on, and had charge and direction of, design and methods of construction, in which work he showed marked ability and originality.

Except for intervals when absence on military service took him away from his engineering work, he continued his connection with Jacobs and Davies to the time (January, 1921) when his illness necessitated his retirement from active business. From 1913, until the time of his retirement, he was a Director, and, from 1917, Vice-President, in the organization of Jacobs and Davies, Incorporated.

In his engineering work, he was always a deep and well-read student of the principles and practice of engineering, and constituted in himself a vast fund of information in all lines of professional work. In a professional capacity, he took an active part in the development of the city subway system of the New York Municipal Railway Corporation and made a special study of the subject of city transportation. He received the George Stephenson Gold Medal from the Institution of Civil Engineers (London) in 1913, for his paper entitled "City Passenger Transportation in the United States." He also contributed to the Society, in 1914, a paper on "Subaqueous Highway Tunnels."* If a single expression could be used which would best describe his personal character, that word would be "faithful."

Col. Snyder was, at all times, deeply interested in military affairs and for a great many years devoted a large part of his spare time to the study of military science and practice. As a young man, he served in all ranks, from private to Captain, in the National Guard of Pennsylvania. When the United States declared war with Spain, he took a commission with the 12th Pennsylvania Volunteer Infantry, and during part of that campaign, he was Assistant Chief Engineer of the 2d Army Corps. When the World War broke out in 1914, he became deeply interested in what he believed to be the inevitable participation by the United States in the war. On November 19th, 1915, he read a short paper before the New York Railroad Club on the subject of "The Railroads and National Defense", which he considered of vital importance to the prosecution of a successful war. He took a great deal of personal interest in this subject, to the extent of obtaining the co-operation of Gen. Leonard Wood, U. S. A., and other prominent military men who discussed the paper

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 252.

and urged on the people the necessity of building up the strength of the National Guard regiments in anticipation of probable interference. The result of his activity in this line was that he was offered a commission in the 22d Regiment, N. G. N. Y., and although he had no desire to go back into the National Guard, he felt that his duty called him to the colors and to a participation in the activities which had arisen due to the war. The Mexican Border troubles of 1916 involved the call of this regiment into active service, and he proceeded, as Lieutenant, to the Mexican Border, serving there from June, 1916, to January, 1917, receiving a commission as Captain while on the Border.

As soon as the United States entered the World War in 1917, he volunteered for active service with the 22d Engineers and assisted in organizing the 102d U. S. Engineers, answering the President's call on July 15th, 1917. He was drafted into the Federal Service on August 5th, 1917, and sailed for overseas duty in May, 1918. He served with 27th Division, A. E. F., and was on duty with the American Commission to Negotiate Peace, in January and February, 1919, in France and Italy. He returned to the United States as Major, U. S. Engineers, and was honorably discharged in April, 1919. Thereafter, he became Major, Engineer Section, Officers Reserve Corps. In the re-organization of the 22d Engineer Regiment, N. G. N. Y., he took an active part as Lieutenant-Colonel, under Col. Thomas Crimmins, and on the latter's retirement in April, 1920, he became Colonel of the regiment. This commission he took with the expressed understanding that he could retire as soon as the regiment was completely Federalized. This he carried out, retiring from the command in October, 1920, when, at his own request, he was transferred to the Reserve List, N. G. N. Y.

His service in the Army, and the regard in which he was held by his fellow officers and men, is particularly well stated in the following letter of Col. Crimmins:

"* * * I want to write you a few lines about my thoughts of George Snyder. I want you to consider them as coming from one who served under and with him. It was only for a brief period that I served as his Colonel. I started in the regiment looking up to George, and I finished that way.

"From February, 1916, when I enlisted in Company G, 22d Regt., N. G. N. Y., under Lieut. George D. Snyder, until April, 1920, when I retired as Colonel of the regiment, I was very closely identified with George Snyder. I observed him as an enlisted man and as an officer. Of George Snyder, I have often said that I never knew a better engineer soldier. Intelligent and studious, he mastered all problems that came before him. Brave and self-sacrificing, he carried out all orders given him, shunning no responsibilities, loyal and conscientious in all his acts, he gave all he had to his country.

"George Snyder will always live in the hearts and minds of all who served with him in the war."

Col. Snyder also served, after his discharge from the Army, as Secretary of the Military Affairs Committee of Engineering Council.

His last illness was protracted and probably originated in his overseas service in the Army, during which time he suffered from shell shock and gas. He

passed away at the home of his sister, at Jersey Shore, Pa., on October 21st, 1921.

Col. Snyder was married in 1901 to Miriam Ellen Harrison, of Williamsport, and is survived by three children.

He was a member of many professional and patriotic societies, including the Institution of Civil Engineers, American Railway Engineering Association, American Institute of Consulting Engineers, Sons of the Revolution, and Society of the War of 1812.

Col. Snyder was elected an Associate Member of the American Society of Civil Engineers on November 6th, 1895, and a Member on September 4th, 1901.

HORACE EDWARD STEVENS, M. Am. Soc. C. E.*

DIED FEBRUARY 14TH, 1922.

Horace Edward Stevens was born in Underhill, Vt., on December 14th, 1848, his father, Luther M. Stevens, coming from Wellesley, Mass., and his mother, Mary A. Catlin, from Burlington, Vt. His early days were spent on a farm and about a small saw-mill operated by his father at the foot of Mount Mansfield in his native State. In 1862, his parents moved to Burlington, in order to give their children (three sons and three daughters) the better educational advantages which that city afforded. He was graduated from the Burlington High School in 1866, and in the fall of that year entered the Engineering Department of the University of Vermont, from which, with Henry H. Douglas, he was graduated in 1870. They were the first to receive the degree of Civil Engineer from that institution. Like many other boys of those days, Mr. Stevens worked his way through the University, earning his board and lodging by doing chores about the home of his landlady, and his spending money during vacation periods by teaching school.

In September, 1870, he went to St. Paul, Minn., and secured a position in the Engineering Corps of the Northern Pacific Railway Company, remaining until October, 1873. During 1870 and 1871, he was on the location and construction of the lines of that railway between Northern Pacific Junction (now Carleton) and Moorhead, Minn. The following year, and in the early part of 1873, his work took him along the lines of that road, through what was then the Territory of Dakota, to the Missouri River. In March, while in camp on the James River, he was ordered, with a party of six men and teams, to drive with the greatest speed across country to the projected crossing of the Missouri, to stake out claims and secure "squatters' rights," for the benefit of the Railroad Company, before the arrival of a party of land speculators, already on their way. The race was a hard one, through snow and blizzards for seven days, first, one party forging ahead, then the other—both arriving at the site of the proposed crossing in the early morning of April 1st, the engineers on foot, their horses worn out and their conveyances abandoned. In the gray of the dawn three of them drove the claim stakes, while the others, with rifles in hand, held the "enemy" at bay. These stakes marked the beginning of Bismarck, the capital city of North Dakota.

In June, 1873, Mr. Stevens was transferred to an expedition detailed to make a survey west of the Missouri River. The engineering party consisted of Gen. Thomas L. Rosser, in charge; A. O. Eckelson, Reconnaissance Engineer; Montgomery Meigs, M. Am. Soc. C. E. (later, U. S. Engineer at Keokuk, Iowa), third in charge; Hubert W. Reed, Transitman; H. E. Stevens, Head Leveler; E. T. Winston, Second Leveler; and F. G. Winston, Third Leveler. A detachment of Government troops under Gen. Stanley, consisting of about 900 Infantrymen, about 800 of the Seventh Cavalry under Col. (later, Gen.) Custer, 100 Indian scouts, and a supply train, accompanied them.

* Memoir prepared by L. S. Oakes, Assoc. M. Am. Soc. C. E.

Starting from Fort Abraham Lincoln, across the river from Bismarck, the party went westward, following closely the subsequently located line of the Northern Pacific Railway, to the confluence of Cedar River with the Yellowstone, about 13 miles above the present site of Glendive, Mont. Crossing the Yellowstone, they continued westward to Pompey's Pillar; thence, northerly, across the Divide to the Musselshell, and down that stream; thence, eastward, to the Yellowstone, crossing near Cedar River, and back to Fort Lincoln, having made an average progress of about 15 miles per day of completed survey, with a maximum of 25 miles. The news of the failure of Jay Cook and Company, the financiers and backers of the Northern Pacific, which attended the panic of 1873, threw the members of the party into a state of mind which was only relieved on their arrival at the headquarters of the Railroad Company, at Brainerd, Minn., where, to their great relief, they found salary checks in full awaiting them for their season's services.

From 1873 to 1881, Mr. Stevens was in the employ of the United States River and Harbor Department. His first work was in connection with the preservation of St. Anthony Falls at Minneapolis, Minn., where a tunnel was constructed from bank to bank under the crest of the cataract, which later was filled with concrete, thus preventing the disintegration of the soft sandstone bed of the Mississippi at that point and halting the recession of the Falls. Later, he made surveys of the Mississippi River between Grand Rapids and St. Cloud, Minn., of the Yellowstone from its mouth to near Miles City, Mont., and of the Missouri from Bismarck to a point near Helena, Mont. He was in camp with a party on Cow Island on the Missouri River, near Judith, Mont., when, in 1877, Chief Joseph and his band of Nez Perces Indians, on the warpath, crossed that stream. Neither party knew of the presence of the other, and no one was molested.

In 1882 and 1883, Mr. Stevens became interested in and was employed by Robinson and Cary, dealers in machinery and contractors' supplies, in St. Paul, Minn. During 1884 and 1885, he did his first contract work—a section of the aqueduct for the St. Paul Water-Works, and some Government harbor work on the Mississippi at Stockholm, Wis.

In 1886, he became Superintendent for Shepard, Siems, and Company, on the construction of the Great Northern Railway between Devil's Lake and Minot, N. Dak. In 1887 and 1888, he was employed as Superintendent of Construction of the "Soo" Line, near Escanaba, Mich., for Henry and Balch. From 1889 to 1893, he built a reservoir and a gas tank in Duluth, Minn., as well as a sewer, water-works, and the London Road at Lakeside, in that city. In 1894 and 1895, he was Water Commissioner of the City of St. Paul, and, in 1896, he constructed water-works at Superior, Wis., and Brainerd, Minn. During 1897, he was President of the Water-Works Company at Bozeman, Mont.

In the early part of 1898, he was selected by the late Lyman E. Cooley, M. Am. Soc. C. E., Engineer, of Chicago, Ill., as one of a committee of engineers and contractors to go over the proposed Nicaragua Canal route for the purpose of investigating and determining the possibilities and structural

feasibility of that project. They left New York City, in January, going first to Panama, where the French were still operating, thence to Nicaragua, where they traversed the line of the proposed canal across that country to the Pacific.

In 1898, he began his association with Winston Brothers, Contractors, of Minneapolis, Minn. During that year and the one following, he was in charge for that Company of a grade revision on the Northern Pacific Railway between Jamestown and Bismarck, N. Dak.; in 1900, of construction on the Cleveland, Lorraine and Wheeling Railway, between Elkhart and Grafton, and between Elyria and Lorraine, in Ohio; and, in 1901, of grading and yard improvements on the Chicago and North Western Railway, at South Omaha, Nebr. When Winston Brothers was incorporated in 1902, as Winston Brothers Company, Mr. Stevens became a Stockholder and Director, and from 1914 to 1920, was Vice-President of the Corporation. In addition to other services for that Company, he had charge of additional work on the Cleveland, Lorraine and Wheeling, between Columbia and Navarre, and between Canal Dover and Uhrichsville, in Ohio; track depression in Milwaukee, Wis., and a short line through Sheboygan, Wis., of the Chicago and North Western; construction of the Indiana Harbor Railway, between St. John and Kentland, Ind.; and a change of line on the Chicago, St. Paul, Minneapolis and Omaha Railway between Minneopa and Lake Crystal, Minn. He also had much to do, from 1907 to 1909, on the reconstruction of the Northern Pacific between Garrison and Missoula, and on a portion of the new Pacific Coast Line of the Chicago, Milwaukee and St. Paul Railway, in Montana, and, later, on a portion of the new line of the Chicago and North Western between Wyeville and Milwaukee, Wis.

He had been in failing health for ten months previous to his death which occurred on February 14th, 1922, at his home in St. Paul, Minn.

Mr. Stevens was married on December 12th, 1893, to Mrs. I. M. Armstrong of St. Paul, who died in 1912. He was a member of the Chicago Engineers' Club, of the Engineers' Society of St. Paul, and, during 1919, was President of the Minnesota Joint Engineering Board.

He was interested in and well informed on many lines of thought outside his profession, had traveled extensively in Europe and the Orient, as well as in his own country, and kept in touch with the progress of events and of developments in the world at large. He was extremely careful and painstaking in whatever he undertook and, although he was on time in his work, he estimated thoroughness and accuracy as of greater importance than speed.

He was known to his business associates as a man of sterling character, of the highest integrity, of pleasing personality, and of excellent business judgment. Although his tendency, due largely to heredity and early environment, was in the direction of extreme conservatism, Mr. Stevens was one who had assimilated the "association idea", and who could and did submerge his individual opinion in business matters in the composite conclusions of the group of his co-workers.

He was, however, unusually self-contained and reserved, which characteristics resulted in his having few intimates. Those who were admitted to terms of intimate friendship recognized in that intimacy a fine privilege, a source of pleasure and profit, through which "he, being dead, yet speaketh."

Mr. Stevens was elected a Junior of the American Society of Civil Engineers on November 1st, 1876, and a Member on April 4th, 1888.

GEORGE SYKES, M. Am. Soc. C. E.*

DIED AUGUST 21ST, 1919.

George Sykes was born on April 5th, 1881, in New York City, where he received his early education.

He began his engineering experience in 1901, as a Leveler in the employ of the Guayaquil and Quito Railway Company for a period of about nine months, before completing his technical education. He then returned to school and, in June, 1903, was graduated from the School of Applied Science of New York University with the degree of B. S. in Mechanical Engineering.

Before his graduation Mr. Sykes entered the employ of the United Electric Light and Power Company, of New York City, as an Inspector on electric transformer building work, and immediately after his graduation continued in the same capacity with the New York Edison Company where his construction experience included the building of office buildings and storehouses in New York City and the erection of a large coal storage plant with a river bulkhead at Edgewater, N. J.

In July, 1907, Mr. Sykes organized a corporation for general construction work and for ten years conducted a very successful contracting business. His field was confined largely to New York City and vicinity, but his most important operation was the building of "Vizcaya", the residence of Mr. James Deering, at Miami, Fla.

When the United States entered the World War, he was commissioned as Captain in the Engineer Officers' Reserve Corps, on July 12th, 1917, and attended the training camp as a student at American University, Washington, D. C. He was ordered to France, in September, 1917, and served as Executive Officer for the Chief Engineer of Line of Communication, as Chief of the Administration Section, in the office of the Director of Construction and Forestry, Service of Supplies, and as Engineer Officer in Chief, Divisional Area Construction, Advance Section, A. E. F. He was promoted to the rank of Major, Engineers, U. S. A., on July 12th, 1918, and after his return to the United States, he was honorably discharged at Washington, D. C., on January 21st, 1919.

Maj. Sykes' patriotism resulted in a tremendous sacrifice to himself and his family which, while it may be true of many other engineers, is none the less deserving of mention; he gave up a prosperous contracting business and spent sixteen months in France away from his wife and children. His military ambitions were probably inherited from his grandfather who was Maj.-Gen. George Sykes, Commander of the 5th Army Corps, Army of the Potomac.

After his discharge from the Army, Maj. Sykes resumed his position as President of the George Sykes Building Company in New York City, and was actively engaged in business until a week before his death on August 21st, 1919, after an operation at the Flushing Hospital. He is survived by his mother, his wife, and three children.

* Memoir prepared by E. M. Van Norden, M. Am. Soc. C. E.

He was an active member of the Bayside, N. Y., Yacht Club and the Delta Phi Fraternity.

Maj. Sykes was elected a Junior of the American Society of Civil Engineers on May 1st, 1906, an Associate Member on December 4th, 1907, and a Member on May 12th, 1919.

HARRY ELSTNER TALBOTT, M. Am. Soc. C. E.*

DIED JANUARY 31ST, 1921.

Harry Elstner Talbott, the son of John Litler and Sarah Elstner Talbott, was born in Cincinnati, Ohio, on July 4th, 1860. He attended the common schools of his native city, and, later, was graduated from the Engineering Division of the University of Cincinnati.

Mr. Talbott's early experience was in railroad engineering, first with the Kentucky Central and the Cincinnati, Lebanon, and Northern Railroad Companies. In 1883, he was advanced to the position of Division Engineer of the Northern Pacific Railroad Company. Four years later, he joined the Engineering Staff of the Elgin, Joliet and Eastern Railroad Company, and afterward went with the Chesapeake and Ohio Railroad Company.

Going to Dayton, Ohio, in 1892, Mr. Talbott opened a Contracting Engineer's Office and soon built up a substantial business. In 1899, he met Mr. Francis H. Clergue, of Sault Ste. Marie, and this meeting was the beginning of a long association and of a great series of extensive engineering projects which have made the name of Mr. Talbott suggest the marvelous development of this great section of country. The discovery of the Helen Mine at Michipicoten, Ont., Canada, 100 miles north of the "Soo", with its rich iron deposits, made it necessary to build the Michipicoten Branch of the Algoma Central Railroad. With a force of 1 200 men, against the doubts of nearly all who contemplated the giant task, Mr. Talbott built the 100 miles of road in that wild, rough country on schedule time. The material, men, and supplies had to be taken back to Michipicoten before the close of navigation, and the work of construction was largely done during the severe winter.

Bridges, ore docks, and other important structures were also built, all to the great surprise of many who observed the wonderful achievement with admiration. Enduring the hardships of the winter with his men, living in a shack or "wigwam", as they called it, near the work, Mr. Talbott endeared himself to the workers, who loved "The Chief" for his comradeship and genial nature.

In the years following, he constructed the greater number of the buildings of the Algoma Steel Corporation and many of the power dams in St. Mary's Rapids, afterward adding several large units to the steel plant and erecting other extensive buildings. His skill at designing, for accomplishing great tasks with dispatch and thoroughness, for handling men and problems, placed his name on the roster of the great builders of the Continent.

In 1911, Mr. Talbott became interested in pulp and paper. The Lake Superior Paper Company was organized by him and Mr. George H. Mead, Mr. Talbott being elected to the Presidency. This beginning led to his becoming a great figure in the pulp and paper world. In recent years, he had held large interests in The Spanish River Pulp and Paper Company,

* Memoir prepared by George Bancroft Smith, Esq., Dayton, Ohio.

The Ontario Paper Company, Limited, The Mead Pulp and Paper Company, The Mead Fibre Company, The Peerless Paper Company, and others.

As he progressed in his several lines of endeavor, he became a dominant figure in the business world. As President of the City National Bank and the City Trust and Savings Bank, of Dayton, Mr. Talbott assumed and held a place among the financial leaders of Southwestern Ohio.

His versatility did not permit of confinement to a restricted line of activity, and he became equally prominent in the manufacturing field. He was one of the founders of The Dayton Metal Products Company, makers of munitions, and of The Dayton-Wright Airplane Company, which organization built nearly 4 000 airplanes during the World War. These two companies were taken over by the General Motors Corporation, in which Company Col. Talbott became a prominent figure. In these enterprises, he was associated with his son, Harold E., and Mr. Charles F. Kettering, the inventor of the Delco starting, ignition, and lighting system for automobiles and the Delco-Light farm lighting outfit.

In 1913, he achieved a great work following the disastrous flood in Dayton, when he assumed the formidable task of cleaning up the wreckage. In recognition of this work, Governor Cox bestowed on him the honorary title of Colonel.

All who knew Col. Talbott were touched by his charming home life. In June, 1887, he was married to Miss Katherine Houk, the daughter of the Hon. George W. Houk, afterward Congressman from the Third District of Ohio. Nine children were born to them, all of whom are living. They are Harold E. Talbott, Mrs. George Shaw Green, Mrs. A. B. Hilton, Nelson Talbott, Mrs. George H. Mead, Mrs. Thomas Hilliard, Miss Lilah, Miss Katherine and Miss Margaret Talbott. His beautiful estate, "Runnymede", in Oakwood, a suburb of Dayton, has been the scene of many great social events, as well as the place of cheer and blessing to neighbors, friends, neighborhood children, and many who have shared the hospitality of Col. and Mrs. Talbott in a thousand ways.

Outstanding men are a great asset to a people. Those who look straight, think sanely, act nobly, live wholesomely, are the arbiters of their own careers and fortunes and an inspiration to thousands who come within the sphere of their activities. How we delight to meet a real fellow. His handshake into which he puts his soul instead of his shadow, his manly pose, his keen sensibilities, his alert mind, all combine to make us realize his potentialities. Such a man was Harry Elstner Talbott, and when he died on January 31st, 1921, it was the close of a splendid career.

As an engineer, capable of great achievements; as a manufacturer, ranking among the foremost; and as a thinker and planner, whose equal is seldom found, he was known throughout the United States and honored because of his ability and likable personality.

Some men can be told how to do a thing and shown how it may be accomplished. Others can conceive a project, provide the ways and means for its consummation, and bring it to a full fruition. Harry Elstner Talbott was that type. He loved big tasks. He thrived on hard jobs. He delighted to carry to

completion what others looked on as impossible. He made one think of "the poor fellow who did not know the thing could not be done, so he just went ahead and did it." Many substantial monuments of his skill and thoroughness stand in various parts of the United States defying time, the elements, and whatever else may assail them.

His generous benevolences were bestowed in an unostentatious way, and countless instances of his tenderness, his humanity, and his philanthropy are known only to the immediate participants. He was a lover of the hunt and of all manly outdoor sports. He rode well, knew the great woods and the haunts of the deer and the moose, and many trophies tell the story of his steady aim.

His ability to judge men was marked, and he used to good advantage his fund of common sense, which was ever apparent. He was resourceful and a man of keen perception, quick decision, and mature judgment.

Col. Talbott was elected a Member of the American Society of Civil Engineers on June 6th, 1900.

WARREN CHAMBERLAIN TUDBURY, M. Am. Soc. C. E.*

DIED MAY 18TH, 1921.

Warren Chamberlain Tudbury, the son of the late John Thomas and Sarah Lizzie (Tibbets) Tudbury, was born in Salem, Mass., on November 14th, 1877.

In 1900, he was graduated from the Massachusetts Institute of Technology with the degree of S. B. in Civil Engineering, and immediately obtained a position with the New York Central and Hudson River Railroad Company. For the next eleven years, with the exception of two brief periods, he remained with this Company and had much to do with the design and execution of important betterments of its lines, including the elimination of grade-crossings, grade revisions, and marine terminals.

From March, 1911, to February, 1912, Mr. Tudbury held the position of Assistant Engineer on Railroad Valuation in the Bureau of Engineering Statistics and, under the direction of George F. Swain, Past-President, Am. Soc. C. E., had charge of bridge calculations and compilations.

From February, 1912, to September, 1913, he was Designing Engineer on important grade-crossing elimination work under the direction of C. M. Spofford, M. Am. Soc. C. E.

In 1914, Mr. Tudbury went to Los Angeles, Calif., to make his home. He there opened an office and carried on a successful engineering practice until late in 1916, when he entered the Government Service in the Bureau of Yards and Docks, at Washington, D. C. His experience in railroad engineering was of great benefit in connection with many of the large projects carried out by the Navy Department during the World War.

In June, 1918, he was transferred to the Engineering Force of the Navy Yard, at Mare Island, Calif., with the title of Expert Aide. Shortly after his transfer to Mare Island, his health began to fail. Notwithstanding his illness, however, he bravely and without complaint gave the Government faithful and valuable service until within a few weeks of his death, which occurred at his home in Berkeley, Calif., on May 18th, 1921.

He is survived by his widow, who, before her marriage in 1914, was Ethel Putnam Wheeler, of Salem, Mass., by a daughter, Patricia Breed Tudbury, and by his mother and sister.

On his mother's side, Mr. Tudbury was descended from some of the earliest and most prominent New England families, families which were identified with the history of the Colonies, the Revolution, and the War of 1812. It is, therefore, only natural, perhaps, that he should have had so great an interest in the early history of the Colonies and in the genealogy of the prominent Colonial families. He collected and arranged a vast amount of genealogical data which he left to the Society of the Sons of the Revolution in Los Angeles. He also left, for future publication, a manuscript giving an account of the adventures of the privateer, *The Grand Turk,* which sailed from Salem under the command of his great-grandfather, Holten Johnson

* Memoir prepared by A. W. Earl, Assoc. M. Am. Soc. C. E.

Breed, during the War of 1812. At the time of his death, he was working on a book entitled "The History of the Copp Family."

The early history of the United States was the most important interest of his leisure hours, but he was also a great lover of birds and a student of their ways. He delighted in photographing them in the various stages of their development, and some of his photographs have been published.

Mr. Tudbury belonged to the Society of Colonial Wars, the Society of the Sons of the Revolution, the Society of the War of 1812, the California Genealogical Society, the American Philatelist Society, and the American Ornithologists' Union.

His great friendliness and gentle ways made for him many friends among those with whom he came in contact, and he will be greatly missed by those that were so fortunate as to know him.

Mr. Tudbury was elected an Associate Member of the American Society of Civil Engineers on June 1st, 1909, and a Member on March 12th, 1918.

GEORGE HERBERT WEBB, M. Am. Soc. C. E.*

DIED NOVEMBER 3D, 1921.

George Herbert Webb, the son of Col. George Webb and Emma (Alder) Webb, was born in Dubuque, Iowa, on March 5th, 1860. Both his father and mother came from Muncy, Pa., where his grandfather had acquired a considerable estate in the early part of the Nineteenth Century. In 1850, his father went West, but after the Civil War he returned to the East and was prominently identified with the construction and operation of various railroads, and with the steel industry. The family made their home in Williamsport, Pa., and, in 1876, moved to Johnstown, Pa.

Mr. Webb was educated at the Pennsylvania Military College, at Chester, Pa. He entered the Sophomore Class in Engineering in September, 1877, and was graduated from this institution, then known as the Pennsylvania Military Academy, as a Civil Engineer with the degree of C. E., on June 10th, 1880.

After his graduation, Mr. Webb entered railroad work as Rodman on the Baltimore and Ohio Railroad. In 1881, he tested steel for the Brooklyn Bridge, and, in 1883, was appointed City Engineer of Johnstown, Pa. Later, he was Assistant Engineer of the Cambria Iron and Steel Company.

From the fall of 1885 to the spring of 1888, Mr. Webb was with the Chicago, Burlington and Quincy Railroad Company as Transitman, Locating Engineer, and Division Engineer on construction, and, in 1888, served as Division Engineer in charge of the construction of parts of the Seattle, Lake Shore and Eastern Railroad (now a part of the Northern Pacific System).

In the spring of 1889, he went to Chile, with the North and South American Construction Company. This Company had a contract with the Chilean Government for building and equipping between 600 and 700 miles of railroad, and Mr. Webb was in charge of the construction of one of the branch lines in the southern part of the Republic. The entire work was stopped later by the outbreak of the Balmaceda Revolution.

On leaving Chile, he went to Peru and, in 1891, was appointed Division Engineer and Superintendent of Construction of the Summit Division of the Central Railway of Peru, formerly the Aroya Railroad. The construction of this Division was a notable piece of work. The line crosses the main range of the Cordilleras de los Andes 15 657 ft. above sea level, and has heavy grades, high curvature, and many tunnels.

In 1893, Mr. Webb returned to the United States and engaged in private engineering practice until 1897, when he was appointed Chief Engineer of the Cincinnati, Georgetown, and Portsmouth Railroad. This road was a small property in charge of Mr. Ralph Peters, and although his associations were pleasant, Mr. Webb wished to study maintenance on larger roads and, at the end of two years, resigned his position to enter the service of the Cleveland, Cincinnati, Chicago and St. Louis Railway Company.

* Memoir prepared by William H. Sellew, Esq., Ann Arbor, Mich.

During 1899 and 1900, he was Supervisor of Track on the latter road and on the Chicago and Alton Railroad, and, in 1901 and 1902, Engineer in Charge of Construction of the Missouri Pacific Shops at Baring Cross, Ark.

Mr. Webb entered the service of the Michigan Central Railroad Company in 1903 and had charge of relocating the main line between Kalamazoo and Lawton, Mich., a work of some magnitude. He was promoted successively to be Division Engineer, Assistant Chief Engineer, and, in 1905, was appointed Chief Engineer. He occupied the latter position until his death, with the exception of about two years, from 1917 to 1919, when he was in military service in France.

The Detroit Terminal of the Michigan Central Railroad is probably the most prominent monument to Mr. Webb's engineering ability. It may be included with such structures as the Grand Central and the Pennsylvania Terminals, in New York City, and the Kansas City Union Depot, as among the four or five largest projects of this character which have been completed within the last decade.

Mr. Webb served in various important military positions in the American Army during the World War, attaining the rank of Colonel. He was awarded the Distinguished Service Medal,* and also the French Ordre de l'Etoile Noire (Officier) by Presidential decree of September 24th, 1919.

On July 6th, 1917, he was appointed Lieutenant Colonel in the Sixteenth (Railway) Engineers. The regiment landed in France on August 29th of that year and took charge of the construction of the Advance Depot at Is-sur-Tille.

About April 1st, 1918, Colonel Webb was placed in charge of a number of railway and camp projects in the vicinity of Nevers. This position soon developed into that of Section Engineer of the eastern half of the Intermediate Section. In this capacity, Colonel Webb had charge of the American engineering projects in about one-seventh of the area of France, extending from the City of Bourges to the Swiss and Italian borders. Among the projects carried on under his direction were the large hospital plants at Mesves-Bulcy, Mars-sur-Allier, and Allerey, with a combined maximum capacity of 72 000 patients.

The Nevers Cut-Off, from an engineering point of view, was the most difficult railway construction by the American Expeditionary Forces. It required a large yardage of embankment and the construction of a pile and steel truss bridge over the Loire River, 2 200 ft. long. Other railway projects were the engine terminal at Marcy, and the car repair shops at Nevers. Warehouses, camps, camp hospitals, and laundries, were constructed and re-organized at various centers, and at Vichy, Clermont-Ferrand, Chatel Guyon, Royat, and Pougues-les-Eaux, groups of resort hotels were taken over and adapted to hospital purposes. At the center at Vichy alone more than 75 000 patients were treated.

All this work was carried on subject to continuous wartime interference with the supply of men, materials, equipment, and transportation. Colonel Webb's inspiring and driving personality stirred the officers and men in the organiza-

* General Orders No. 59, War Department, Washington, 1919, p. 33.

tions under him, and by his resourcefulness, initiative, and skill, he overcame all obstacles and completed these difficult projects with marked success.

On September 27th, 1918, he was promoted to the rank of Colonel and in April, 1919, returned to the United States and resumed his duties as Chief Engineer of the Michigan Central Railroad.

In the fall of 1920, Colonel Webb's health showed signs of failing, and in the following spring an operation was performed without, however, obtaining the hoped for relief. He died on November 3d, 1921, at the home of his sister, Mrs. G. L. Osgood, Jr., near Boston, Mass.

Colonel Webb was a man of strong personality and sterling qualities. He was a thorough student and possessed keen insight, and his work showed in a high degree evidence of original and self-reliant thought. His interesting experiences and unusual personal charm made him a delightful companion, and his death will be long felt by the many who had the privilege of knowing him.

In January, 1905, he was married to Jessie Lawrence, who died in May, 1907. They had many congenial tastes, and his home life afforded pleasant relief from the close application he was in the habit of giving to the duties of his profession. Her untimely death was much to be regretted.

On June 17th, 1914, the Board of Trustees of the Pennsylvania Military College conferred on Colonel Webb the honorary degree of Master of Civil Engineering, in recognition of his high professional attainments and his worth as a man and citizen.

He was a member of the American Railway Engineering Association, and served the Association as Director and Chairman of the Committee on Rules and Organization. He was the first Commander of the Detroit Chapter of the Military Order of the World War, a member of the Sixteenth Engineers Post Veterans of Foreign Wars of the United States, and of the American Legion.

Colonel Webb was elected a Member of the American Society of Civil Engineers on February 1st, 1893.

WILLIAM GLYDE WILKINS, M. Am. Soc. C. E.*

DIED APRIL 12TH, 1921.

William Glyde Wilkins, the son of Alvin and Charlotte Glyde Wilkins, was born in Pittsburgh, Pa., on April 16th, 1854. With his parents, he moved to Detroit, Mich., in 1855, and attended the public schools in the latter city.

In September, 1872, he entered the Rensselaer Polytechnic Institute, Troy, N. Y., but after one year's stay at that institution, he left to enter the employ of the Munsing Iron Company, Lake Superior. In 1875, Mr. Wilkins became connected with the Engineering Department of the Pennsylvania Railroad Company at Pittsburgh, remaining there until September, 1876, when he returned to Troy to complete his engineering education at Rensselaer, from which he was graduated in 1879 with the degree of Civil Engineer.

Mr. Wilkins immediately entered the service of the United States Government and was assigned to the corps that was making a hydrographic survey of the Mississippi River in the vicinity of Fulton, Tenn. He remained there until June, 1880, when he entered the Engineering Department of the Pennsylvania Railroad Company as Assistant Engineer of Construction, with headquarters at Philadelphia, Pa. In July, 1887, he left the service of this Company and opened an office in Pittsburgh for private practice. On January 1st, 1890, he associated with himself George S. Davison, M. Am. Soc. C. E., under the firm name of Wilkins and Davison. On Mr. Davison's retirement from the firm on January 1st, 1900, the business name of the concern was changed to The W. G. Wilkins Company, Mr. Wilkins associating with himself Wilber Macaulay Judd, M. Am. Soc. C. E., and several of his more prominent employees, and this association continued to the time of his death, on April 12th, 1921.

While with the Pennsylvania Railroad Company, Mr. Wilkins laid out and had charge of the construction of the branch line from Phœnixville to Fraser, Pa.; the Charles Street Depot at Baltimore, Md., which has been replaced recently by a new structure; the Duquesne Freight Station at Pittsburgh; and the stone arch bridge of the main line of the Pennsylvania Railroad over the Conemaugh River at Johnstown, Pa.

One of his earliest engagements in private practice was the design of the first steel head-frame for a coal mine shaft known to have been built in the United States. Although his practice covered every branch of civil, mining, and mechanical engineering, his intense interest in coal mining and coke making led him to specialize in that class of work, and it is undoubtedly true that he was interested in the design of more mining and coke plants than any other engineer in America.

In the first fifteen years of his private practice Mr. Wilkins laid out and constructed a great number of industrial railways in the Pittsburgh district and also built a large mileage of electric street railways. His intimate knowl-

* Memoir prepared by George S. Davison, M. Am. Soc. C. E.

edge of the coke business caused his appointment, in 1907, as one of the three Trustees of the Estate of William Thaw, Deceased, Coke Trust, owners and lessors of many thousands of acres of coal lands in the Connellsville coke region. He performed the duties of this office to the time of his death.

For a limited period he filled the office of City Engineer of Allegheny, Pa., before that city became a part of Pittsburgh. At that time the city was engaged on a programme of extensive public improvements, which naturally came under his supervision and design.

In addition to his membership in the Society, Mr. Wilkins was a member of the following technical societies: American Institute of Consulting Engineers, American Institute of Mining and Metallurgical Engineers, American Mining Congress, North of England Institute of Mining and Mechanical Engineers, Coal Mining Institute of America, Academy of Science and Arts of Pittsburgh, and the Engineers' Society of Western Pennsylvania. In the latter Society, he had served as a Director in 1890 and 1891, and was President in 1896.

At the time of the formation of the Pittsburgh Flood Commission in 1908, he was appointed a member and served actively on the Engineering Committee. This Committee made an exhaustive study of the water-sheds of the Allegheny and Monongahela Rivers, reporting on a plan for controlling the flood waters of these streams through the construction of impounding reservoirs. Mr. Wilkins gave freely of his time and talent in the preparation of the plans for the report.

As a member of the Chamber of Commerce of Pittsburgh he was most active in the civic affairs of his home city. He served as Director of this body for six years from 1908.

At the session of the Pennsylvania Legislature in 1911, the City of Pittsburgh was granted a new charter, which abolished the plan of entrusting the legislative work of the city to two large bodies of councilmen and placed it in the hands of nine councilmen elected at large. The first body was appointed by the Governor on June 5th, 1911, and Mr. Wilkins was one of those appointed. When his commission expired, he was elected by popular vote, and continued in office until December 31st, 1915.

As an office-holder, his words and deeds always went directly to the point at issue. His policy was to serve the public to the best of his ability and without favor to any one. His engineering knowledge and skill brought about many reforms in the methods of public improvement.

Mr. Wilkins possessed one of the largest private libraries in the City of Pittsburgh, and he was internationally recognized as being the best informed student on the works of Charles Dickens in the world. He delivered many lectures on Dickens and was a member of the Dickens Fellowship. Through the English firm of Chapman and Hall, Limited, of London, he published "Charles Dickens in America", and at the time of his death there was in the hands of his publishers, the Bibliophile Society of America, a new book written by him on "Dickens in Cartoon and Caricature."

As in everything else, Mr. Wilkins was active in religious affairs, having been a member of the North Presbyterian Church, of Pittsburgh, which congregation he served as Trustee for many years. He was married to Sarah Rebecca Simmons at Troy, N. Y., on December 29th, 1880, by whom he is survived.

Mr. Wilkins was elected a Member of the American Society of Civil Engineers on December 4th, 1889, and served as a Director of the Society during 1909, 1910, and 1911.

JOHN WILSON, M. Am. Soc. C. E.*

DIED JUNE 28TH, 1921.

John Wilson was born in Wisbeach, Cambridgeshire, England, on January 23d, 1841. He was graduated from Cooper's Hill Civil Engineering College, in the suburbs of London, in 1865, and the following year was sent by the English Government to India as an Engineer in the Public Works Department.

Later, Mr. Wilson was appointed Assistant Engineer on the Ganges Canal, after which he was transferred to Bengal, India, as Engineer on the construction of the Sone Canal. In 1876, famine broke out in Bengal, and he was sent to the Provincial Branch for famine relief work and was assigned to the District of Dinagepore in the low swampy grounds of the Ganges Valley. There he served until the famine was over and gained the title of Major, by which he was ever afterward known.

In 1882, Major Wilson came to the United States and was engaged as Consulting Engineer on the construction of cableways in Philadelphia, Pa., Chicago, Ill., and Pittsburgh, Pa. In 1892, he went to Texas to take up the survey and design of a hydro-electric power plant near Waco.

In 1894, Major Wilson was appointed Engineer for the Estate of the Corralitos Ranch and Mining Company in the State of Chihuahua, Mexico. During this engagement, he surveyed the ranch of a million acres and planned and designed the improvements relative thereto, which included the building of dams and the laying of miles of water pipe.

In 1899, Major Wilson was engaged by the El Paso Electric Railway Company on the construction of the electric railway system for El Paso, Tex. During 1904, 1905, and 1906, he designed and constructed the irrigation system for the Big Valley Irrigation Company, at Grand Falls, Tex., from which work he went as Chief Engineer and Manager for the Barstow Irrigation Company, at Barstow, Tex. He remained at Barstow until he was appointed a Member of the Board of Water Engineers for the State of Texas in September, 1913. He served on this Board for two terms, after which he retired from active practice and moved to California.

He was a man of strong character and of great determination, adhering strictly to his own ideals, regardless of public comment, and, in his death, the people have lost a man who helped to make the world better.

Major Wilson was elected a Member of the American Society of Civil Engineers on March 2d, 1915.

* Memoir prepared by Vernon L. Sullivan, M. Am. Soc. C. E.

JOSEPH WOOD, M. Am. Soc. C. E.*

DIED MARCH 4TH, 1922.

Joseph Wood, the son of Isaac H. and Elizabeth (Cooper) Wood, was born in Haddonfield, N. J., on June 5th, 1846. He was educated in Philadelphia, Pa., and was graduated in 1864 from the Polytechnic College of the State of Pennsylvania as a Civil Engineer.

Mr. Wood began his railroad career in October, 1864, as Rodman on the construction of the Connecting Railway, Philadelphia, Pa. In 1867, he was appointed Assistant Engineer, having entire charge of the work which included the bridge over the Schuylkill River, near Fairmount Park. In 1865, he also had charge of finishing a tunnel on the Junction Railroad, Philadelphia, and building an extension of the Pennsylvania Railroad to Greenwich Wharves, Delaware River, including the construction of three wharves. During 1868, he made surveys for the location of the Columbia and Port Deposit Railroad in Pennsylvania and Maryland, and located and built part of the South Mountain Railroad from Carlisle to Pine Grove Furnace, Pa.

In 1869, Mr. Wood entered the service of the Northern Central Railway Company as Resident Engineer. In this capacity, he was engaged on the construction of the engine houses and shops at Marysville, Pa., and also the rebuilding of the Dauphin Bridge over the Susquehanna River. In May, 1872, he was appointed Resident Engineer of the Baltimore and Potomac Railroad, then being constructed, and Engineer of the Northern Central Railway.

In May, 1878, Mr. Wood was appointed Assistant Engineer in the Motive Power Department of the Pennsylvania Railroad at Altoona, Pa. This was the beginning of his career with the Pennsylvania Company with which he was connected until his death and which he served so ably. In December, 1881, he became Superintendent of Motive Power for the Lines West of Pittsburgh, and in November, 1887, was appointed General Superintendent of Transportation.

Mr. Wood became General Manager in March, 1890, and was elected Fourth Vice-President in January, 1896. He was afterward promoted to the offices of Second and First Vice-Presidents, successively, having been elected as First Vice-President in January, 1907.

In April, 1913, he was elected a Member of the Board of Directors of the Pennsylvania Company, and served also as a Director of the Pittsburgh, Cincinnati, Chicago and St. Louis Railway Company, and of many of the subsidiary companies in the Pennsylvania System West of Pittsburgh. Mr. Wood severed his connection with the Lines West of Pittsburgh in February, 1914, after nearly fifty years of service. He had served as a representative of the Pennsylvania Company on the directorates of other companies in which the System was interested, and at the time of his death on March 4th, 1922, he was a Member of the Board of Directors of the Norfolk and Western Railway Company and Chairman of its Finance Committee.

* Memoir compiled from information furnished by J. J. Turner, Vice-Pres., Penn. Lines W. of Pitts., Pittsburgh, Pa., and on file at the Headquarters of the Society.

Although most of the active years of his life were spent in the service of the Lines West of Pittsburgh, he thoroughly understood the affairs of all the lines connected with the Pennsylvania System.

Having begun his service in the early days of railroading, and having had experience in all its branches, Mr. Wood was an authority on railroad management, and also in the fields of commerce, industry, and finance. He was firm in his convictions, but not prejudiced, always willing to listen courteously to the opinions of others. He was loyal, conscientious, and efficient in his work, through which qualities he attained success in his official career. Mr. Wood was greatly respected and esteemed by his friends and business associates, and his loss will be felt by all with whom he came in contact.

In October, 1874, he was married to Miss Jennie E. Boas, who died on December 14th, 1913.

Mr. Wood was elected a Member of the American Society of Civil Engineers on April 1st, 1874.

ROBERT STUART ARMSTRONG, Assoc. M. Am. Soc. C. E.*

DIED JULY 15TH, 1918.

Robert Stuart Armstrong, the son of Richard and Annie Armstrong, was born on February 20th, 1874, at Hamilton, Ont., Canada, where he received his early education. He was graduated from the Hamilton Collegiate.

In 1890, he was employed by Messrs. Jennie and Mundie, Architects, of Chicago, Ill., and while in this position he took up engineering under the late James D. McKee with whom he went to Dayton, Ohio, in 1893 (with the firm of Williams and Andrews, Architects), and to St. Louis, Mo., in 1895. While in St. Louis, Mr. Armstrong acted as Engineer for the School Board Architect, and also assisted the late Professor J. B. Johnson, M. Am. Soc. C. E., of Washington University, in the preparation of material for his book "Materials of Construction", making many of the drawings and plates for this work. Mr. Armstrong also studied perspective drawing and water colors, in which he became very proficient, and his services in this line of work were frequently employed.

During this period of Mr. Armstrong's life his future career shaped itself, but there evidently was a struggle within him as to whether he would follow his early training and inclination toward Architecture or the Profession of Engineering, in the choice of which he undoubtedly was greatly influenced by his teacher and friend, Mr. McKee, a very far-sighted and progressive engineer.

Mr. Armstrong was employed by the Koken Iron Works of St. Louis until the fall of 1898, when he was engaged by the Jackson Iron Works of New York City, and, subsequently, with Post and McCord until 1900, when the plant of the latter firm was acquired by the American Bridge Company. Shortly after the formation of the Bridge Company, Mr. Armstrong was transferred to the Trenton Plant, at Trenton, N. J., where he remained until February, 1903, when he was transferred to the New York Office, as an Estimator and Designer, and made a specialty of designs for office buildings, theatres, armories, and similar structures.

In July, 1904, he was made Engineer of the Brooklyn Plant of the American Bridge Company, in charge of the Engineering Department. In May, 1915, he became Manager of the Brooklyn Plant, which position he held until September, 1915, when he resigned to become Contracting Manager of Milliken Brothers, and when the plant of this firm was purchased by the Downey Shipbuilding Company he remained with that Company as Works Manager.

In 1918, Mr. Armstrong became Construction Manager for the Carolina Shipbuilding Company of New York City and Wilmington, N. C. Subsequently, he became Fabricating Manager, and left New York City to take up his permanent abode at Wilmington, N. C., where he met his sudden and untimely death, only one week after his arrival there.

Mr. Armstrong was married on June 28th, 1916, to Miss Elizabeth Egginton, of Brooklyn, N. Y., who survives him.

* Memoir prepared by H. Fougner, Assoc. M. Am. Soc. C. E.

"Bert" Armstrong, as he was familiarly known to his friends, was highly regarded both as engineer and man. As a young man, his proficiency made his services greatly sought after, and when, in later years, he was called on to direct the work of other men, his ability to convey his efficiency to others gained for him the high respect and confidence of his employers.

His sunny nature and sterling qualities made for him a host of friends, and the affectionate friendship of his employers and of the many whose work he was called on to direct is a splendid tribute to his memory.

Mr. Armstrong was elected an Associate Member of the American Society of Civil Engineers on February 6th, 1907.

PAUL JONES BEAN, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 25TH, 1919.

Paul Jones Bean was born in Woodville, Tex., on March 17th, 1884. He received his preparatory education in the common schools of Texas, and was appointed to the U. S. Naval Academy, as Midshipman, on September 2d, 1902. On his graduation, in 1906, he was selected for the Corps of Civil Engineers, U. S. Navy, and was commissioned Assistant Civil Engineer, with the rank of Lieutenant (Junior Grade) in that Corps on April 27th, 1906. He was ordered to the Rensselaer Polytechnic Institute, at Troy, N. Y., for professional instruction, and was graduated from that institution in 1908 with the degree of Civil Engineer.

Immediately following his graduation from Rensselaer, Lieut. Bean was assigned to duty as Inspector for electrical machinery at the works of the General Electrical Company, at Schenectady, N. Y., and during this duty completed the regular Apprentice Instruction Course of that Company. On April 9th, 1910, he was assigned to duty as Assistant to the Public Works Officer, at Norfolk, Va., and, in August, 1911, having completed the necessary examination, he was transferred to the grade of Civil Engineer, with the rank of Lieutenant. In August, 1913, he was detached from the Navy Yard at Norfolk, and ordered to the Naval Station, at Honolulu, Hawaii, as Assistant to the Public Works Officer at that station. During the period of this duty, Mr. Bean contracted tuberculosis, and in November, 1914, was ordered to the Naval Hospital, at Mare Island, Calif., for treatment and observation. After six months' leave, he was sent to the Naval Hospital at Las Animas, Colo., and on March 16th, 1916, was transferred to the retired list.

After his retirement, and treatment in various places, Mr. Bean's health improved to such an extent that he was able again to engage in active work, and he accepted the position of Secretary and Treasurer of the Laurel River Logging Company. It was while on an inspection of this Company's operations at Runion, N. C., that he contracted the disease which ultimately resulted in his death at the Mission Hospital, in Asheville, N. C., on January 25th, 1919.

Mr. Bean was married on February 19th, 1908, to Miss Ethel M. Phillips, of Troy, N. Y., who, with four children, Ethel, Paul, Virginia, and Fields, survives him.

By the death of Mr. Bean, the Corps of Civil Engineers of the United States Navy and the Civil Engineering Profession generally have lost a young man of exceptional promise. He had a genius for organization, and in the comparatively short period of his active service, he had acquired a reputation as an expert in engineering office methods as applied to costs and cost analyses. He was a man of sterling character and was genuinely loved by his subordinates as well as his superiors.

* Memoir prepared by Leonard M. Cox, Capt., C. E. C., U. S. N., M. Am. Soc. C. E.

At the time of his death, he was a member of the Pafraets Dael Club, of Troy, N. Y., the Mohawk Country Club, of Schenectady, N. Y., and the Army and Navy Club, of Washington, D. C.

Mr. Bean was elected a Junior of the American Society of Civil Engineers on May 2d, 1911, and an Associate Member on December 31st, 1913.

HAROLD INGERSOLL BELL, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1921.

Harold Ingersoll Bell was born at Canton, Ill., on August 16th, 1880. He prepared for college at the Bay City, Mich., High School. After one year at the University of Michigan, he entered Cornell University in 1901, from which he was graduated in Civil Engineering.

Since 1906, Mr. Bell had been associated with the H. P. Cummings Construction Company of Ware, Mass. In 1912, he was made Manager of the Portland Office of the Company, which was opened that year, which position he held until his death.

In this capacity, Mr. Bell had charge of the construction of a number of large plants in the State of Maine, the best known of which was the Hydro-Electric Development of the Rumford Falls Power Company, at Rumford, Me.

On April 27th, 1916, he was married to Ellen Morrell Foster, daughter of Mr. and Mrs. T. D. Foster, of Ottumwa, Iowa, who, with two sons, Hugh Foster and Gordon Humphrey, survives him. He is also survived by his mother, Mrs. Mary E. Bell, of Ithaca, N. Y., and two sisters.

Mr. Bell was elected an Associate Member of the American Society of Civil Engineers on August 31st, 1915.

* Memoir prepared by Henry S. Huntington, Esq., New York City.

ANSON MORSE BLENUS, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 5TH, 1922.

Anson Morse Blenus was born at Worcester, Mass., on July 2d, 1873, where he received his common and High School education. Later, this was supplemented by special courses at the Mechanics Institute and at Columbia University, New York City.

His early days were spent in the employ of contractors as Time Clerk, Material Clerk, and Foreman.

During the Spanish War, Mr. Blenus served aboard a gunboat of the United States Navy as Assistant Engineer. In the following year, he went to South Africa and, during the Boer War, served with the British forces, in the Royal Engineers, developing and operating the emergency water supply for the garrison during the siege at Ladysmith.

In the following year, Mr. Blenus returned to New York City and entered the employ of the George Mulligan Company, which firm sent him to Capetown, South Africa, where he was in charge of its work for several years.

In 1909, he entered the employ of W. J. Jones, Civil Engineer, and took part in the power development and the construction of the underground conduit system for the City of Reading, Pa. During this period, he also had charge of the underground system for the New York, Westchester and Boston Railroad Company.

From 1912 to 1914, Mr. Blenus was employed in the capacity of Designer in the offices of the Pearson Engineering Corporation, and took part in the Sorocaba (São Paulo), Brazil and Ebro (Spain) Projects.

In 1914, he entered the Engineering Department of the Interborough Rapid Transit Company, on the Manhattan Elevated Improvement, and in 1917, he joined the Construction Division of the Texas Company as Designing Engineer.

He entered the employ of the Foundation Company, in 1918, as Designer and, in 1920, joined the Canadian Engineering Agency, Incorporated, in a like capacity. He held this position at the time of his death from toxic poisoning supervening a fever which he had contracted while in South Africa.

Mr. Blenus was married in October, 1905, to Clara Mabel Baker who, with a daughter, Madeline L., survives him.

He was a man of pleasing personality and social disposition, who made friends easily and was liked and respected by all his associates.

Mr. Blenus was elected an Associate Member of the American Society of Civil Engineers on July 6th, 1920.

* Memoir prepared by J. R. Berberick, Esq., New York City.

JAMES GIBBONS BROWNE, Assoc. M. Am. Soc. C. E.*

DIED APRIL 25TH, 1921.

James Gibbons Browne, the son of John T. and Mollie Bergin Browne, was born on December 16th, 1886, at Houston, Tex. He received his preliminary education in the public and parochial schools at Houston, and entered the University of Texas in the fall of 1904, from which he was graduated in June, 1908, with the degree of Civil Engineer.

From 1908 to 1910, Mr. Browne was employed in various municipal improvements in the City of Houston and in some of the smaller cities of Texas, and, in 1910, he engaged in practice for himself, undertaking municipal work, sewage, pavements, bridges, and similar contracts.

Mr. Browne made rapid progress and, at the time he was stricken with what later proved to be a fatal illness, was regarded as one of the most promising young engineers in the construction field in Southern Texas. In the fall of 1915, following an attack of ptomaine poisoning, a serious and complicated illness developed. In spite of the services of several specialists, and most conscientious efforts to find a cause and remedy for this malady, Mr. Browne grew steadily worse. He passed most of the year preceding his death on April 25th, 1921, in a state of coma.

Mr. Browne's grasp of the business side of construction work, and his large fund of common sense, promised for him a most successful and useful career. His kindly and generous personal qualities cannot be forgotten by those who knew him best.

On November 24th, 1910, he was married to Annie Marguerite Casperson, of Houston, Tex., who, with one child, Annie Marguerite, survives him.

Mr. Browne was elected an Associate Member of the American Society of Civil Engineers on May 6th, 1914.

* Memoir prepared by J. C. Stevenson, M. Am. Soc. C. E.

JOSEPH MILLER BURKETT, Assoc. M. Am. Soc. C. E.*

DIED APRIL 26TH, 1921.

Joseph Miller Burkett, the son of Joseph Miller Burkett and Ida Adelle Pinney Burkett, was born in Salt Lake City, Utah, on March 30th, 1880. His father was long associated with the pioneer banking firm of McCornick and Company, in Salt Lake City, but, in 1882, moved with his family to Hailey, Idaho, where he opened the First National Bank of that city. Mr. Burkett's early childhood was spent in Hailey which, at that time, was an active and prosperous mining camp, so it was only natural that his first engineering efforts were in connection with the mining industry.

In 1890, he entered the Ogden Military Academy, at Ogden, Utah. After attending the Academy for three years he went to Boise, Idaho, and entered the public schools of that city. While in Boise, Mr. Burkett resided with his uncle, Mr. James A. Pinney, one of the well known business men and early pioneers of Idaho. On the completion of his public school education, he entered Oberlin College, in Oberlin, Ohio, where he remained for three years. While at Oberlin, he spent his summer vacations selling books and following similar pursuits common to the more ambitious students of that day. Desiring to engage in the Engineering Profession, Mr. Burkett then entered the Leland Stanford, Jr., University, in California, and while there was engaged during his summer vacations in various capacities with field crews on surveys for the Santa Cruz Railroad.

After leaving Stanford, Mr. Burkett returned to Idaho and began the practice of engineering. At first, he was engaged principally in mining work, but owing to the rapid irrigation development of Southern Idaho about this time, he became occupied with reclamation work almost exclusively and spent several years in active field work of various kinds. Although a considerable part of his practice was located in Idaho, he also did more or less work in Montana and Nevada and, at one time, was interested in a large irrigation project in Texas.

In 1911, Mr. Burkett entered the employ of the State of Idaho as Carey Act Engineer, serving under A. E. Robinson, State Engineer, and continuing in the employment of the late Frank King who succeeded Mr. Robinson. In the capacity of Carey Act Engineer, he made various investigations and generally supervised the construction work on practically all the principal Carey Act projects in Idaho. In 1915, he resigned his position with the State and entered into private practice, locating at Twin Falls, Idaho. While maintaining his office in that city, he conducted a general engineering business, having had charge of considerable municipal work for the various communities that were rapidly building up in Southern Idaho.

When the United States entered the World War, Mr. Burkett at once volunteered and was appointed a Captain of Engineers. After a short period of training at Vancouver, Wash., he was immediately sent overseas with the

* Memoir prepared by W. G. Swendsen, Assoc. M. Am. Soc. C. E.

116th Engineers, with which he served as Captain throughout the war. He was located at various places in France for nearly two years, and was honorably discharged after the Armistice was signed.

On his return from France, Mr. Burkett again took up the practice of his Profession, with Boise, Idaho, as his headquarters. His practice while in Boise was confined almost altogether to the supervision of irrigation developments throughout the State, having served as Chief Engineer on some five or six projects in various stages of completion. He also acted as Consulting Engineer to the Commissioner of Reclamation of Idaho in connection with various irrigation problems coming before the Department of Reclamation, and, acting under the Commissioner, he conducted engineering investigations incident to several important river adjudication suits, the principal among which were those relating to the Portneuf and Pahsimeroi Rivers.

Owing to his extensive knowledge of irrigation developments in Idaho, and his broad experience in irrigation matters generally, Mr. Burkett was often called on for expert testimony in connection with litigations which occurred from time to time, and in this capacity was invariably regarded as an engineer of the highest ability and integrity.

He died on April 26th, 1921, at Pocatello, Idaho, after a hurried operation for appendicitis, to which he submitted while on a field trip, and was buried at Boise. He was not married, and is survived by his mother who lives in Hailey, and an only sister, Mrs. T. M. Starrh, of Boise. He was a member of the Masonic order and of the Benevolent and Protective Order of Elks.

Mr. Burkett was elected an Associate Member of the American Society of Engineers on May 28th, 1912.

FREDERICK WALLIS DAGGETT, Assoc. M. Am. Soc. C. E.*

DIED MAY 10TH, 1921.

Frederick Wallis Daggett, the son of the late William H. Daggett and Nellie I. Daggett, was born in Boston, Mass., on July 26th, 1877. He spent his boyhood days in the place of his birth and received his early education in the public schools of Boston. In 1895, he entered the Lawrence Scientific School, Harvard University, and was graduated in the Class of 1899.

After leaving college, Mr. Daggett's first engagement was with the Fore River Shipbuilding Company of Quincy, Mass., where he was employed as Draftsman for two years. He then became associated with the United Coke and Gas Company of New York City, as Designer and Resident Engineer on the erection of by-product coke-oven plants and gasworks, with which Company he remained for four years. Subsequently, for several years, he was employed, respectively, by the Ransome Concrete Machinery Company, the Ransome and Smith Company, and E. L. Phillips and Company, all of New York City, designing and superintending the construction of reinforced concrete buildings.

In September, 1907, Mr. Daggett began work as Superintendent of Construction of a section of the Catskill Aqueduct, near Peekskill, N. Y., for the Thomas McNally Company, and, later, its successor, John J. Hart. From October, 1909, to April, 1912, he was associated with Fred T. Ley and Company, of Springfield, Mass., as Engineer on the construction and design of filtration plants, during which time he had charge of the construction of a 3 000 000-gal. mechanical filter for the City of Burlington, N. J., and a 5 500 000-gal. plant for the Ayer Mills of the American Woolen Company, at Lawrence, Mass.

During 1912 and the early part of 1913, Mr. Daggett was employed by the Merrill-Ruckgaber Company of New York City, as Engineer in charge of the construction of a 6 000 000-gal. filter plant for the City of Cumberland, Md., and on the completion of this work, he was engaged by Johnson and Fuller as Resident Engineer on a 30 000 000-gal. filter plant at Trenton, N. J. When this work was completed in 1914, Mr. Daggett was appointed by the City Government as Superintendent of the Plant, which position he held at the time of his death, on May 10th, 1921.

In 1914, he was married to Ernestine Giddings, of Waltham, Mass. As a result of this union, one son—Jerome D. Daggett—was born. In August, 1915, Mrs. Daggett died, and on May 26th, 1919, he was married to Frances B. Glasgow, of Burlington, N. J., who, with his mother and son, survives him.

Mr. Daggett was a profound student and a thorough and efficient engineer. He was a man of high character and possessed personal qualities which earned for him a wide circle of friends.

He was a Charter Member of the Engineers' Club of Trenton and served for a number of years as a member of its Board of Governors. He was also a member of the Harvard Club of New York City.

Mr. Daggett was elected an Associate Member of the American Society of Civil Engineers on May 1st, 1907.

* Memoir prepared by N. A. K. Bugbee, Assoc. M. Am. Soc. C. E.

WILLIAM JAMES DAVIS, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 2D, 1921.

William James Davis was born in Lachine, Que., Canada, on June 26th, 1884. He was the son of Thomas Davis, of Lachine, and Mary Sproul Davis, a native of Maxville, Ont., Canada. He attended the public schools in Montreal West and the Montreal High School, and his technical education was obtained in a special class conducted in Montreal by W. Chase Thomson, M. Am. Soc. C. E., who is now a Consulting Engineer in that city.

In May, 1905, Mr. Davis began work as a Templet Maker for the Locomotive and Machine Company of Montreal, which Company had at that time just begun the fabrication of structural steelwork as an adjunct to its tank work. To him, a job was not something to be "held down", but an opportunity for work, and this optimistic energy, which was one of his characteristics, showed itself in the templet shop from the very first.

During the fall of 1907, he left the Locomotive Company to enter the Drawing Office of the Dominion Bridge Company's Head Office at Lachine. In a short time, he proved his worth in this field also and was promoted to be a Checker. In this capacity, Mr. Davis had considerable experience in structural steelwork for railway and highway bridges, swing bridges, office buildings, mill buildings, conveyor trestles, wharf sheds, elevators, theatres, churches, and miscellaneous structures.

In 1913, the Dominion Bridge Company decided to open a Drawing Office at its new branch in Winnipeg, Man., and Mr. Davis was chosen to take charge of this new Drafting Department. Within two months, he was made Construction Engineer of the Winnipeg Plant and acted as Assistant to the Western Manager, George E. Bell, Assoc. M. Am. Soc. C. E., whose headquarters were at Winnipeg. It was the formative period in the Company's business in that place and was a time of keen competition. Probably no one except those on the ground realized the tireless energy with which Mr. Davis threw himself into every detail of the Winnipeg Plant. The results proved the good judgment which had been shown in his appointment.

Directly following the entry of Great Britain into the World War, a period of reorganization took place, and, in 1915, Mr. Davis returned to the Head Office of the Dominion Bridge Company, at Lachine, as Assistant Works Manager, a position for which he was well suited by his experience in the shop, in the office, and on the Executive Staff at Winnipeg. In his new capacity, he showed his ability to make things move, and his example was an incentive to those with whom he came in contact in the works.

During the summer of 1919, Mr. Davis left the Dominion Bridge Company to accept a position as Engineer with the Wayagamack Pulp and Paper Company, at Three Rivers, Que. In this capacity, he acted as Chief Draftsman, Plant Engineer, and Assistant to the President. Owing to a decline in the paper business, the Engineering Staff was reduced appreciably, but the Com-

* Memoir prepared by F. P. Shearwood, M. Am. Soc. C. E.

pany knew Mr. Davis' worth and retained his services. He designed and carried out considerable extensions to the machine-room and other buildings for the Company, his work including the approval of contractors' drawings, the design of foundations, and the superintendence of the construction of new buildings and of repairs to existing conveyors and bridges.

In the latter part of August, 1921, he took a well earned holiday, with some companions, in the timber limits of the Wayagamack Company, on one of the tributaries of the St. Maurice River. On September 2d, he went out with a guide on a small body of water called Lake Cutaway, and, through a slight accident, the canoe was capsized. It is thought that although he was not a good swimmer, he could have saved his own life had he not been anxious to save that of the guide also. The guide survived, but Mr. Davis sank in 40 ft. of water. His body was recovered after two days, sent to Three Rivers, and, from there, to Montreal where the funeral took place. Besides his widow and two daughters, Mr. Davis is survived by his father and mother, three brothers, and a sister.

On September 6th, 1911, he was married to Miss Florence Nutter, of Montreal. The marriage proved a singularly happy one, and their home was known to many friends on account of the marked hospitality of the host and hostess.

To all his business acquaintances and friends, Mr. Davis was known as "Bill". He was a man among men, a good sport in the very best sense of the word, and he could and did look every one straight in the eye. He was active in almost every sport, but particularly so in curling and bowling. Already a challenge shield has been presented to the Three Rivers Bowling Club by the Bowling Club at Montreal West, to be known as the "Bill Davis Shield". It is only one evidence in many of his work for good clean sport. His life—thirty-seven years—seems short, but it cannot be called incomplete. His consistent and intense record is an example not surpassed by long life and is a challenge to all.

As early as 1912, he became connected with the Canadian Society of Civil Engineers, now the Engineering Institute of Canada.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on August 31st, 1915.

THOMAS GEORGE ELBURY, Assoc. M. Am. Soc. C. E.*

DIED JULY 6TH, 1921.

Thomas George Elbury, the son of Edward and Jane (Simmons) Elbury, was born at Bristol, England, on February 24th, 1862. His family for many generations had been engaged in the manufacture of pottery and clay products. During his boyhood, besides going to school at St. Luke's, Bedminster, Bristol, he was being educated to carry on the established business of his ancestors.

He showed little inclination for this work, however, and resolved to fit himself for a profession. At the age of fifteen, he started out to earn his own living, and by working half the day, and going to school the other half, he laid the foundation for his future career.

After considering the opportunities offered young men in his native country, Mr. Elbury decided that a greater future awaited him in the United States; in accordance with this decision, he left England for the United States in August, 1883, and, two weeks after his arrival, he settled in Cleveland, Ohio. Working by day and going to school at night, he continued his studies and, in spite of many obstacles, he qualified for his chosen profession, Civil Engineering.

In 1885, Mr. Elbury moved to Kansas and was appointed Assistant Civil Engineer of Barber County. A few years later, he was made Deputy Surveyor of Kingman County. In 1895, he was elected County Surveyor of Reno County, which position he held until 1905.

From 1902 to 1909, Mr. Elbury served as City Engineer of Hutchinson, Kans., where his work included the design of the Hutchinson Drainage Canal. In addition to other duties, he contracted and made preliminary surveys for the Northern Oklahoma Railway in 1902, and, in 1904, he was employed as Chief Engineer of the Gulf, Hutchinson, and Northwestern Railroad.

His last engineering work was the construction of the dam for the Columbia and Okanogan Rivers at Kennewick, Wash., and the dam at St. Maries, Idaho, after which he retired from active engineering work.

For two years previous to his death, Mr. Elbury was associated with his son, in the publication of a technical paper, the *Big-4-Railroad Record,* in San Francisco, Calif.

In April, 1920, he visited his old home in England. Shortly after his return to San Francisco, he became seriously ill with a malady that developed into pneumonia, from which he died on July 6th, 1921.

On August 21st, 1889, Mr. Elbury was married to Alice A. McKinnis, of Nashville, Tenn., who, with a son, Edward John Elbury, survives him.

He was a man of excellent character and was held in high esteem by his many friends and associates.

Mr. Elbury was elected an Associate Member of the American Society of Civil Engineers on May 6th, 1908.

* Memoir compiled from information furnished by Edward John Elbury, Esq., and on file at the Headquarters of the Society.

FRANKLIN LINCOLN GIBBONEY, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 20TH, 1921.

Franklin Lincoln Gibboney was born at Wytheville, Va., on July 15th, 1882. He received his early education in the public schools of his native town, and was graduated in 1904 from the Virginia Polytechnic Institute, Blacksburg, Va., with the degree of Bachelor of Science in Civil Engineering.

In May, 1904, he accepted the position of Assistant Engineer of the City of Roanoke, Va., in which capacity he had charge of street and sewer improvements for that city.

In September, 1909, Mr. Gibboney was elected City Engineer of Roanoke, and was re-elected as such for nine consecutive terms, until 1918. He was, also, for a short time afterward, connected with the State Highway Commission of Virginia.

In 1919, he was appointed as Material Engineer of the State Highway Commission of North Carolina, which position he held at the time of his death at Charlotte, N. C., on November 20th, 1921.

He is survived by his wife and one son, William, and by his parents and two brothers and two sisters. He was of a happy disposition and drew around him a wide circle of friends who mourn his loss.

Mr. Gibboney volunteered for service and was commissioned a First Lieutenant in the Engineering Department of the American Expeditionary Forces, having been ready for duty when the Armistice was signed.

He was a consistent and faithful member of the Protestant Episcopal Church, and was prominent in Fraternal circles. He was a member of the American Association of Engineers and of the North Carolina Society of Civil Engineers.

Mr. Gibboney was elected an Associate Member of the American Society of Civil Engineers on October 31st, 1911.

* Memoir prepared by W. S. Fallis, M. Am. Soc. C. E.

RICHARD TUGGLE GOODWYN, JR., Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 8TH, 1921.

Richard Tuggle Goodwyn, Jr., was born at Montville, near Aylett, Va., on June 23d, 1892, the son of Richard Tuggle Goodwyn and Sally (Aylett) Goodwyn, both of Virginia.

Trained in the preparatory schools of St. Charles Military Academy and Rugby Military School at St. Charles, Mo., he entered the University of Georgia in 1909 and was graduated in 1913, with the degree of Bachelor of Science in Civil Engineering and with a notably fine scholastic record.

As an undergraduate, Mr. Goodwyn held commissions as Second Lieutenant, Company F (1912), and Captain, Company D, in the University Battalion. He was Secretary-Treasurer of the Engineering Society, a member of the Sine and Tangent Club, Editor in Chief of the *Engineering Annual,* and Vice-President of the Senior Class.

In the fall of 1913, Mr. Goodwyn began his professional career as Civilian Assistant in the United States Engineer Corps, at Brunswick, Ga., on river and harbor work; and shortly afterward accepted service as Valuation Engineer with the Interstate Commerce Commission, with headquarters at Chattanooga, Tenn.

He remained in this service, with rapid promotion, until the United States entered the World War. His first effort to enlist as a volunteer in the Army was barred by an unfavorable medical report; but, by further effort, he secured a transfer as a Private into the Fifth Regiment, Engineers, U. S. A., and was shortly afterward commissioned as Second Lieutenant of Engineers.

Having received his discharge from the Army in December, 1918, Mr. Goodwyn resumed his position with the Interstate Commerce Commission. In January, 1920, he was appointed by the State Highway Department of Georgia as Division Engineer in charge of the State highways in the Eighth Congressional District, with headquarters at Athens, Ga. He filled this position with exceptional efficiency until his tragic death on November 8th, 1921. While in the discharge of his duty, he was accidentally killed, his automobile having overturned from an embankment, on a dark rainy night, at the Alcovy River Bridge, near Covington, Ga.

He was a member of the American Association of Engineers and the Society of Military Engineers. He was associated with the Chattanooga Branch of the American Legion, and was a faithful and active member of Emmanuel Church, Athens, Ga.

Young in years, Mr. Goodwyn had, to a singular degree, won the confidence of his professional associates and of the public with whom his large responsibilities brought him in contact.

Under the most difficult post-war conditions, charged with the detailed planning and construction of about 500 miles of main State highways, involving large expenditures of funds arising in co-operation from the Federal Government, from various counties, and from the State of Georgia, he estab-

* Memoir prepared by C. M. Strahan, M. Am. Soc. C. E.

lished a reputation for sound engineering and executive judgment, for untiring energy and singleness of purpose, for scrupulous integrity and effective performance which exemplify the finest and best ideals of the Engineering Profession.

Distinguished by a fine intellect and high personal character, by a pleasing, quiet, but firm bearing, by a habit of thorough mastery of facts and details, by an unswerving sense of duty, by a sympathetic understanding of men and power of leadership, by an unassuming but controlling religious spirit, Mr. Goodwyn leaves a record of professional and personal worth which keenly emphasizes the sadness of his untimely death and the loss to his chosen profession.

Mr. Goodwyn was elected an Associate Member of the American Society of Civil Engineers on September 12th, 1921.

JOHN EDWARD GRADY, Assoc. M. Am. Soc. C. E.*

DIED MAY 19TH, 1921.

John Edward Grady was born in Kenosha, Wis., on May 10th, 1874. He received his common school education in Chicago, Ill., finishing two years of High School and a short course in a business college in December, 1890.

At the age of sixteen, he went to work as a Timekeeper for the Kelly Construction Company, General Contractors, thus beginning his engineering education in the school of experience. In April, 1893, he was engaged as a Rodman on the location of the Chicago Sanitary and Ship Canal and continued in the employ of The Sanitary District of Chicago until June, 1906, when he left of his own accord to join the Great Lakes Dredge and Dock Company.

During the thirteen years (1893-1906) of his employment with the Sanitary District of Chicago, Mr. Grady progressed steadily, and grew in character, ability, and attainments. In 1894, he was variously employed on canal construction. In 1895 and 1896, he was engaged in the Computing Department on preliminary and final estimates, progress charts, and construction reports. In 1897, he was put in charge of the Computing Department and, in 1899, he was given charge of the construction of the Chicago River By-Pass under the Pennsylvania Railroad Company's properties at the Chicago Union Station. In 1901, he was put in charge of bridge construction across the Chicago River and, during the ensuing five-year period, he completed the building of five bascule bridges at an aggregate cost of $1 125 000.

It was during this period that the writer had the privilege of serving under Mr. Grady and of learning, by daily intercourse, his fine qualities of mind and heart. He possessed, in a marked degree, the qualities of leadership as evidenced by the devotion of his subordinates, the friendly admiration of his equals, and the respect and confidence of his superiors.

During the thirteen years (1906-1919) of his employment with the Great Lakes Dredge and Dock Company, Mr. Grady occupied a position of increasing responsibility. His first work for this Company consisted of the engineering supervision of the extensive dock, harbor, and foundation work for the United States Steel Corporation, at Gary, Ind. While still conducting this work, he was made Division Engineer for the Company, with headquarters at Cleveland, Ohio, and had charge of its extensive interests in the Cleveland Division. He installed the large plant at Cleveland for the manufacture of premoulded concrete piles which were made in large quantities and first used in the extensive ore docks built for the Pennsylvania Railroad Company in that city. He also planned and built the foundations for the High Level Bridge at Cleveland, consisting of the main and approach piers and caissons. Mr. Grady planned and conducted the work on the Pennsylvania Railroad Bridge over the Maumee River, at Toledo, Ohio, on the extensive ore docks for the Cincinnati, Hamilton and Dayton Railroad Company, and on a large coal dock for the Hocking Valley Railroad Company, both at Toledo.

* Memoir prepared by Robert Isham Randolph, M. Am. Soc. C. E.

During the construction of these larger works, he was also in charge of numerous Government and industrial developments on Lake Erie, at Ashtabula, Sandusky, Conneaut, and Lorain, Ohio. At the time of his death, he was a member of the committee of the Cleveland Engineering Society for the straightening of the Cuyahoga River.

In July, 1919, Mr. Grady left the Great Lakes Dredge and Dock Company to become one of the principal partners in the Central Dredging Company, of Cleveland, and just prior to his death, he had organized the Substructure Company of Cleveland.

He was a systematizer of work, and his orderly mental processes were evidenced by the manner in which he planned and directed everything within his jurisdiction. He developed many new and useful methods of construction, and his inventive genius and originality were always at the service of his employers and clients. He was an indefatigable worker, and, although he always had the end in view, nevertheless, he appeared to work for the joy of working.

Mr. Grady was a "self-made man", in the best sense of the word, "self made", but never "self satisfied", for he continued diligent and tireless in making and improving his structure of manhood until his death, which occurred on May 19th, 1921.

On June 8th, 1898, he was married to Miss Alice Archer, of Chicago, who survives him. This partnership was the inspiration of both his business and his private life and the sustaining force which kept him happy and hopeful through the years of his last illness.

His creed and his code were not matters of oral profession, but were evidenced by his daily life and conduct. When once asked to express his rule of life, he replied: "Faithul performance of contract."

Mr. Grady was elected an Associate Member of the American Society of Civil Engineers on December 6th, 1905.

EDGAR MILLER GRAHAM, Assoc. M. Am. Soc. C. E.*

DIED MAY 14TH, 1921.

Edgar Miller Graham was born in Buffalo, N. Y., on June 26th, 1881. He was the son of M. W. Graham, of Franklin, Pa., and Helen R. (Hall) Graham, of Ashtabula, Ohio. He obtained his preliminary education in the public schools, High School, and at Underwood Institute, at Buffalo, with the intention of entering the Massachusetts Institute of Technoogy. Instead of entering a technical school, however, he immediately started to work. He acquired his technical education by studying standard textbooks, with the assistance of technically educated friends. He also completed the Railroad, Municipal, Hydraulic, and Electrical Engineering Courses of the International Correspondence Schools.

In 1900, Mr. Graham began his engineering career as Chainman for an engineering firm in Buffalo, N. Y. His rise was rapid, for during the same year he served successfully as Transitman, Draftsman, and Chief of Party on work for the firm. Most of this work was in connection with the staking out of the large buildings for the Pan-American Exposition and also field engineering in connection with the Dakota and Great Eastern Grain Elevators, which were among the largest in the United States, the latter being one of the first reinforced concrete elevators in Buffalo.

During part of 1901, Mr. Graham was engaged in the Engineering Department on the Eastern Division of the New York Central and Hudson River Railroad. Later in 1901, and until 1903, he was in the service of the Lackawanna Steel Company on the construction of its large plant at Buffalo. During this time, he served as an Assistant in the Estimate Division, as Chief of Party on laying out all kinds of work, and as Assistant Engineer to the Chief Engineer.

Later in 1903, he was in charge of the design and construction of several miles of paving and sewers at Clearfield, Pa., and, in 1904, he was employed by the Harbison, Walker Refractories Company, at Pittsburgh, Pa., in the Sales, Operating, and Mining Departments.

From 1905 to 1908, Mr. Graham was in the service of the Buffalo and Susquehanna Railway Company, first, as Draftsman, then Chief Draftsman, and Assistant Engineer in charge of the office of the Assistant Chief Engineer at Buffalo. In 1907, he was made Assistant Chief Engineer, in charge of the Buffalo Office and of all engineering and construction work on 90 miles of railway from Wellsville, to Buffalo, N. Y.

From 1908 to 1910, Mr. Graham was engaged in private practice at Buffalo. In the latter year, he moved to Oklahoma, and from that time until his untimely death, he maintained an office in Muskogee, as a Consulting Engineer. During this period of eleven years, he was very active in engineering and construction work in Oklahoma and the adjoining States. He made a number of investigations and reports for proposed railroads, acted as Consult-

* Memoir prepared by Milton Leon, Assoc. M. Am. Soc. C. E.

ing Engineer for several towns, and made several valuations of railroad and public service properties.

As Chief Engineer, he had charge of all engineering work of the Muskogee Electric Traction Company for 11 years. In the same capacity, he located the Webbers Falls Railroad and built the first 10 miles, all that was ever built. Mr. Graham was Consulting Engineer and Director of the Cushing Traction Company. In the capacity of Chief Engineer of the Cushing Construction Company, he had full charge of the location and construction of the lines of steam railroad built in the Cushing, Okla., Oil Fields. For the last few years of his life, he had made a special study of highway construction and paving in connection with the "Willite" process of paving.

Mr. Graham met his death in an automobile accident near Muskogee, Okla., on May 14th, 1921. In turning aside to avoid a wagon, his machine plunged into a deep ditch, and he was killed instantly.

In 1907, he was married to Gertrude Thornton, the daughter of Mr. and Mrs. James Thornton, of Wellsville, N. Y., who, with his father, survives him.

Mr. Graham was a Thirty-second Degree Mason, a member of the Shrine, Benevolent Protective Order of Elks, Jovian League, Little Rock Engineers Club, and the American Association of Engineers. As President of the Local Chapter of the American Association of Engineers and as Vice-President of the Oklahoma Section of the Society, Mr. Graham was very active in all work for the betterment of engineering and advancement of the professional engineer.

He was held in highest esteem by his friends and associates, and was one of the best known engineers in the State.

Mr. Graham was elected an Associate Member of the American Society of Civil Engineers on October 5th, 1909.

GISLI GUDMUNDSSON, Assoc. M. Am. Soc. C. E.*

DIED JULY 19TH, 1921.

Gisli Gudmundsson was born at Bordeyre, Iceland, on March 31st, 1862. He attended the Technical Institute, Copenhagen, from 1883 to 1885, and received the degree of Civil Engineer from the Teknisk Loereanstalt, at Trondhjem, Norway, in 1889. At this time, he entered the service of the Norwegian Government, and was engaged in highway construction for two years, having been employed a part of that time on the "Kaiser Wilhelm Way," and on harbor work.

In 1893, Mr. Gudmundsson resigned from this position and came to the United States, settling in Pittsburgh, Pa., which city was his home until his death. For the next three years, he was employed as Draftsman and Estimator with the Shiffler and Keystone Bridge Companies, with the Lake Erie and Ohio River Canal Commission, and with the Pennsylvania Lines West of Pittsburgh. For a short time during this period, he also served as an Engineer on the location of thirty miles of railroad in Guatemala.

From 1896 to 1898, Mr. Gudmundsson was with the late Hermann Laub, Assoc. M. Am. Soc. C. E., on bridge and building design and erection, and, in 1898, he had charge of construction with the Pittsburgh Plate Glass Company. In 1899, he again entered the employ of Mr. Laub, this time as his Principal Assistant. In this position, Mr. Gudmundsson had charge of the design and erection of the Allegheny River Bridges at New Kensington and Highland Park, Pittsburgh, and numerous other bridges and buildings.

In 1903, he entered into business for himself and was engaged in a general civil engineering practice until his death. During this time, he designed and built the bridges for the Indianapolis and Louisville Traction Railway Company; the power house and car barn for the Southern Cambria Railway Company, at Mineral Point; the Woodvale Avenue Viaduct, at Johnstown; the South Fork Bridge, at South Fork; and the highway bridge over the Pennsylvania Railroad at Ebensburg, Pa. He also designed and had charge of the alterations and construction of the new plants of the Mesta Machine Company at West Homestead; the Kittaning Brick and Fire Clay Company, at Kittaning, and the Lockhart Iron and Steel Company, at Pittsburgh, Pa., and made examinations and detailed reports on several bridges over the Beaver River for Beaver County, Pennsylvania. From 1911 to 1917, he acted as Consulting Engineer for the Pittsburgh and Butler Railway Company, during which time he made detailed reports on all the bridges on the line, and designed and built the bridges from Slippery Rock to Grove City, Pa.

At different intervals, he co-operated with the Engineer's Office of Allegheny County, at Pittsburgh, in the design of the Sewickley Bridge over the Ohio River, and in remodeling the 7th and 30th Street Bridges over the Allegheny River, and the 3d and 5th Avenue Bridges over the Youghiogheny River, at McKeesport, Pa.

* Memoir prepared by Vernon R. Covell, M. Am. Soc. C. E.

Mr. Gudmundsson was a careful student of events in the United States and abroad, and took a deep interest in all civic and philanthropic matters. He was of a genial disposition and had many warm personal friends.

He was an active member of the Engineers' Society of Western Pennsylvania and of the Lincoln Club of Pittsburgh. He had no relatives in America.

Mr. Gudmundsson was elected an Associate Member of the American Society of Civil Engineers, on January 3d, 1900.

ARTHUR JOHN HART, Assoc. M. Am. Soc. C. E.*

DIED JUNE 26TH, 1920.

Arthur John Hart, the youngest son of the late Sidney John Hart, was born at Chatham, Kent, England, on December 15th, 1887. His early life was passed in England where he received his education.

In January, 1905, Mr. Hart secured a position as Pupil with the City Engineer of Rochester, England. He quickly advanced to the rank of Assistant on the construction of a concrete reservoir and six miles of electric tramway, and on the re-survey of that city. In September, 1908, he entered the employ of the Indented Bar and Concrete Engineering Company, Limited, in London, England, and spent 3½ years as Assistant on the design and supervision of many large reinforced concrete works, at H. M. Dockyard, Rosyth; the military barracks, at Abbasia, Cairo, Egypt; H. M. New Stationery Office, London; the new Royal Automobile Club, London, and many other similar works.

In May, 1912, Mr. Hart was appointed Chief Representative Engineer in Australia for the Indented Bar and Concrete Engineering Company, Limited, and left England to take up his new position in Sydney. In a short time, he began the practice of his profession as Consulting Engineer and Chief Representative Engineer for the Indented Bar Company. His chief interest lay in reinforced concrete and his progressive methods and ideas, together with his comprehensive knowledge of this building material, quickly placed him at the forefront, as the most active advocate for this method of construction in Australia.

From 1912 to 1920, Mr. Hart regulated and controlled a constantly expanding practice and kindred businesses. His practical criticism and series of lectures before the Institute of Civil Engineers in Sydney were responsible to a great extent for the revision of the municipal by-laws of the City of Sydney relating to construction in reinforced concrete and structural steel. During these eight years, the work under Mr. Hart's charge must have aggregated in value several millions of pounds sterling. Everywhere in Sydney, and throughout the Commonwealth of Australia, his works stand as eloquent and striking testimony to his greatness as a designer and constructor, and as continual reminders to his friends.

Bearing on the earlier period of Mr. Hart's life there is a dearth of information, but from what his friends in Australia know of his fine personality, capacity for noting detail, together with the end in view, and his hunger for further information, it is not surprising to learn that his interest in life was by no means centered in the practice of his profession alone.

He was a member of the Automobile Club of Australia and of the Royal Sydney Yacht Squadron; his car and his yacht *Lowanna,* his hockey, tennis, and other outdoor interests, he enjoyed with wholehearted vigor. For a time, he was Secretary of the Athletic Rifle Club of Sydney. Offering his

* Memoir prepared by Myles J. Dunphy, Esq., Sydney, New South Wales, Australia.

services to his country early in the World War, and having been rejected for the physical defect which eventually proved fatal, he gave a large part of his time to war service work.

It has been said, and there is some truth in the saying, that an engineer is almost wholly devoid of the artistic perception, but Mr. Hart at least was a master constructor, an expert administrator, and a capable art critic.

Mr. Hart extended his help to the young craftsmen and professional cadets at the Sydney Technical College in a free and practical manner; for some years, and up to the time of his death, the prize list never failed of his contribution, and he esteemed it an honor to be allowed to set an examination paper annually in the subject nearest his chosen profession.

He was married in 1916 to Miss Gwendolyn Wood, the youngest daughter of the late Quayle Wood, of Brundah Station, and Mrs. Wood, of Kirribilli Point, Sydney, who, with their two baby daughters, survives him.

Mr. Hart was elected an Associate Member of the American Society of Civil Engineers on October 14th, 1919.

HENRY HARVIE, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 14TH, 1921.

Henry Harvie was born near Birmingham, England, on October 4th, 1873. He received his technical education at University College, London. On the completion of his studies in 1896, he entered the service of the Great Western Railway Company as an Assistant Chief Draftsman at the Swindon Works, where he remained until 1903. Mr. Harvie's ability as a Draftsman and Designer was recognized among those who knew him as being of the highest quality, combining the artistic with the technical to a remarkable degree.

In 1903, Mr. Harvie came to Canada, and joined the Engineering Staff of the Ontario Power Company, at Niagara Falls, Ont., Canada. In 1905, he joined the Designing Department of the J. G. White Engineering Corporation of New York City. He resigned in 1906, to accept the position of Chief Draftsman for the Bush Terminal Company, Brooklyn, N. Y., and, later, was appointed Chief Draftsman of the Long Island Motor Parkway.

From 1908 to 1910, Mr. Harvie was again with the Ontario Power Company as Chief Draftsman, during the extensive additions which were then being made to its plant at Niagara Falls. At that time, this plant was numbered among the largest developments of the world, and the great difficulties to be overcome in both design and construction required engineering talent of the highest quality. In 1910, Mr. Harvie was appointed Assistant to the Chief Engineer of the Lake Superior Power Company, in charge of the design of a hydro-electric power development on the Magpie River in the Michipicoten District, north of Lake Superior, for supplying power to the mines operated by the Algoma Steel Company. On the completion of this work, he joined the Engineering Staff of the Hydro-Electric Power Commission of Ontario, at Toronto, where he was for the most part employed as Chief Draftsman on hydro-electric power plant design.

Latterly, Mr. Harvie was employed by C. H. and P. H. Mitchell, Consulting Engineers, of Toronto. The unfortunate accident which caused his death on October 14th, 1921, took place in the plant of the General Electric Company, at Peterborough, Ont., while he was inspecting a shop test of a generator rotor.

He was married in England to Miss Dora Gaskell who, with three children, survives him.

Mr. Harvie had acquired a broad experience and training, particularly in the hydro-electric field, and had become recognized as an authority on the various features of power-plant layout and design. This, coupled with his natural ability as an engineer, had placed him high in the ranks of his Profession.

* Memoir prepared by G. R. Heckle, M. Am. Soc. C. E.

He became an Associate Member of the Canadian Society of Civil Engineers (now the Engineering Institute at Canada), in 1912, and for some time was a member of the Engineers' Club of Toronto, where he had a large circle of friends.

Mr. Harvie was elected an Associate Member of the American Society of Civil Engineers on May 12th, 1919.

CHARLES RAYMOND LARKIN, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 30TH, 1921.

Charles Raymond Larkin was born in Philadelphia, Pa., on November 6th, 1891. He was educated in the public schools of that city and was graduated from the North East Manual Training School in 1910. He then entered Villa Nova College, from which he received the degree of B. S. in Civil Engineering in 1914, and the degree of Civil Engineer in 1919.

Mr. Larkin began his engineering work with the Union Paving Company, as Superintendent and Highway Engineer, and many sheet asphalt streets, in the City of Philadelphia and its vicinity, were laid under his direction.

In August, 1916, Mr. Larkin was appointed Assistant Engineer in the Bureau of Health, City of Philadelphia, under J. A. Vogelson, M. Am. Soc. C. E., Chief of the Bureau, and, in this capacity, all construction and alterations undertaken in the hospitals and other institutions under the Bureau were done under his supervision.

In 1918, he was given a leave of absence from the Bureau of Health and enlisted in the Army, attending the Training School at Camp Joseph Johnston, Jacksonville, Fla. He was commissioned as a Second Lieutenant in the Quartermaster Corps on December 6th, 1918, but was retired to the Officers' Reserve Corps and again resumed his duties with the Bureau of Health where he remained until his death on August 30th, 1921.

Mr. Larkin was married on November 17th, 1920, to Katharine E. Lochrey, at Jamaica, Long Island, N. Y., and is survived by his widow, his father, and a brother.

He was a man of sterling character and exceptional ability and showed an earnestness and zeal in everything which he undertook that inspired the confidence of his associates and gave promise of a brilliant career. He will always be remembered as one of those rare personalities whose bigness of heart and breadth of spirit endeared him to all.

Mr. Larkin was elected a Junior of the American Society of Civil Engineers on January 14th, 1918, and an Associate Member on June 1st, 1920. He was also a member of the Engineers Club of Philadelphia and the Henry H. Houston Post No. 3 of the American Legion.

* Memoir prepared by H. B. Floyd, Esq., Philadelphia, Pa.

HARRY MILTON LYNDE, Assoc. M. Am. Soc. C. E.*

DIED MAY 17TH, 1921.

Harry Milton Lynde, the son of James, Jr., and Lizzie Estelle (Clifford) Lynde, was born in Chelsea, Mass., on December 27th, 1882. He was educated at the Massachusetts Institute of Technology, having been a member of the Class of 1905, and at the Polytechnic Institute of Brooklyn, from which he was graduated in 1906, receiving the degree of C. E.

During the summer vacation in 1903 and 1904, Mr. Lynde was employed as Rodman by the Metropolitan Park Commission of Massachusetts, and the summer of 1905 was spent as Rodman on the Boston and Albany Railroad.

The greater part of the next two years were spent by him in gathering and compiling data for the National Board of Fire Underwriters and Fire Insurance Companies.

In October, 1907, Mr. Lynde was appointed Assistant Drainage Engineer, Office of Experiment Stations, U. S. Department of Agriculture, which position he retained until May, 1909. During this time, he was engaged principally on drainage surveys and investigations in the Southern States.

From May, 1909, to January, 1913, he served as Assistant Engineer to the New York Board of Water Supply engaged in the construction of the Catskill Aqueduct.

In January, 1913, Mr. Lynde returned to the office of Experiment Stations, U. S. Department of Agriculture, and took charge of the drainage work of the office in North Carolina, which position he held until his death at Raleigh, N. C., on May 17th, 1921.

His most important work while in this position was the planning and conducting of investigations relating to the drainage of agricultural lands. His investigations in regard to the effect of tile drains on the ground-water table, which he discussed in a paper read before the Southern Section of the American Society of Agricultural Engineers, at Lexington, Ky., a short time before his death, was a valuable contribution to agricultural science. During this period, he devoted a considerable part of his time to the study of methods of prevention and control of soil erosion, and many examples of his skill in this work are to be found in North Carolina. In 1915 and 1916, he demonstrated the use of terraces for controlling soil erosion in Missouri, and so successful was his work that a general interest in improvements of this character was aroused.

Mr. Lynde was always a student and had a gift for research. His character was of the finest, and his dealings were always characterized by fine courtesy and integrity.

On June 28th, 1911, Mr. Lynde was married to Amy Louise Edmester, of Massachusetts, who, with a daughter, Marguerite, survives him.

He was a member of the American Society of Agricultural Engineers and the American Association of Engineers.

Mr. Lynde was elected a Junior of the American Society of Civil Engineers, on April 14th, 1911, and Associate Member on April 1st, 1914.

* Memoir prepared by S. H. McCrory, M. Am. Soc. C. E., and Clifford Lynde, Assoc. M. Am. Soc. C. E.

RALPH EWART ROBSON, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 14TH, 1921.

Ralph Ewart Robson was born in Sioux City, Iowa, on June 27th, 1886. His father was John Ewart Robson, an Engineer and Contractor, whose sudden death in 1899 placed on the boy the duties of contributing to the family support and securing an education for himself. Principles of industry and activity had been early inculcated in him by both his parents, and these principles he adhered to with unflagging energy until his death.

The boy attended the Grammar and High Schools in Sioux City, and was graduated from the latter in 1904, as President of his Class. Although he was far from robust as a lad, he found no task too difficult, no labor too arduous, as long as it meant a continuance of his education. After his graduation, he joined a surveying party with the late Brig.-Gen. (then Col.) H. M. Chittenden, U. S. A., M. Am Soc. C. E., and spent the summer and early fall in Yellowstone National Park. He was ill after his return and, in the spring of 1905, his health not being good, he went to California. After more than a year of hard work in various parts of the State, he entered the University of California, from which he was graduated in 1911, with the degree of B. S. in Civil Engineering. He was a member of the Delta Tau Delta Fraternity.

To help out his finances and enable him to continue at college, Mr. Robson went to Alaska in the spring of 1908, with the Fremont-Morse party to establish the boundary between the United States and Canada. While still in college, he wrote and had accepted for publication in various periodicals, articles on the work in Alaska, illustrated with his own photographs.

After his graduation from the University of California, Mr. Robson entered the employ of Sloan and Robson and while with this firm worked on designs for water and sewer systems for various California municipalities. He was also Superintendent or Engineer in Charge of several projects, at Fullerton, Anaheim, Tulare, and other places.

In January, 1914, he became interested in the plan of the Atascadero Colony near Paso Robles and there, under H. T. Cory, M. Am. Soc. C. E., who was Chief Engineer, he stayed two years, becoming Resident Engineer of the project before he left in 1916.

Maintaining the interest in military affairs which began when he was a child, Mr. Robson attended the first Civilian Training Camp held at Monterey in July, 1916. In the late summer of the same year, he went to Utah for the Utah Copper Company, to superintend the construction of a leaching plant. He returned to California and, in January, 1917, went to the Anderson-Cottonwood Irrigation District, where he remained until May of that year, when he entered the First Officers' Training Camp, in the Engineering Section, at the Presidio of San Francisco. The Engineers were transferred to Vancouver Barracks, Washington, to complete their course, and there Mr. Robson

* Memoir prepared by Thomas H. Means, M. Am. Soc. C. E.

was graduated, in August, 1917, with the rank of Captain of Engineers and was at once assigned as Commanding Officer of Company E, 316th Engineers, at Camp Lewis, Wash. He was training troops and preparing for overseas duty until June, 1918, when with the remainder of the regiment, together with all of the 91st Division, he left for France, where his company joined the American Expeditionary Force and received the final training which was to prepare it for active combat work in the front line.

Of Mr. Robson's part in the World War, a fellow officer, Captain Ross Mahon, has the following to say:

"From the day on which Company E was organized, Ralph Robson evidenced the characteristics and attributes which are required for the highest type of American officer. Through the long, hard weeks of the early training period, he applied himself untiringly to the work of building up the most efficient kind of an organization. Night and day he labored to overcome the difficulties which presented themselves. No detail was so small or unimportant that it could be overlooked. * * * All who knew him realized that Ralph Robson was, perhaps, the most sincere and thoroughly in earnest officer of the regiment. * * * Again, for six weeks the hard grind of training continued on French soil and the love and loyal devotion that Captain Robson received from the men and officers under his command, did more, perhaps, than any other one thing to produce an efficient fighting unit from the 250 untrained men who had reported to him twelve months earlier.

"Captain Robson, with his company, took part in the St. Mihiel Offensive and the Meuse-Argonne Offensive, after which the 316th Engineer Regiment, with the 91st Division, was shifted to the Flanders front. On October 31st, and just before his company went into the front line in the Ypres-Lys Offensive in Belgium, Robson was detached from Company E and, although still a Captain, was given command of the First Battalion during the interval when it was actively engaged in combat, from November 1st to November 10th, 1918.

"On November 10th, he received his promotion to Major and from that date until the final mustering out of service in May, 1919, he retained command of his battalion. * * *

"All who knew the man through those two years have only the highest words of praise for Major Robson. His devotion to duty, his absolute sincerity, his love and regard for his fellow men, his square dealing, and his unfailing good nature, will live long in the memories of his comrades in arms."

The award of the Belgian War Cross by the Belgian Government was made to Major Robson for the following services rendered, the citation reading as follows:

"On October 31, 1918, and November 1, 1918, he handled his Battalion in a very efficient manner in the reconstruction of roads and bridges in the vicinity of Waereghem, Belgium, permitting the early advance of the Divisional Trains and Artillery."

After his return from France, Major Robson became Manager of the vineyard of the American Seedless Raisin Company, near Livingston, Calif., where his tireless energy and ambition enabled him to perform his duties in the most satisfactory manner, study and review his French, and take an Extension Course in Viticulture and one in Military Tactics.

In February, 1921, he became associated with the Cope Rand Means Company, in San Francisco, Calif., and made his home in Berkeley. He put

forth all his efforts toward making a place for himself in the engineering world, making every minute count, studying irrigation engineering, writing a paper on the "Work of the American Divisional Engineer in France," and renewing college friendships and making new ones.

In September, 1921, he was appointed Resident Engineer of the Tule and Baxter Creek Irrigation Districts, in Lassen County, California. While performing his duties for the Irrigation Districts, he was killed instantly, on October 14th, 1921, when the car he was driving ran into a guy wire which had been stretched from a haystack across a public road.

The news of his sudden death came as an appalling shock to his host of friends in the United States, France, and Belgium. It seemed a queer turn of Fate that a man so vital, so forceful, who had faced death on a battle front and come through unscarred, should lose his splendid life in such an accident.

His magnificent physique, won by his outdoor work and his clean living, made him an ideal figure of the best type of American soldier, and to those who saw him in his unform, he will be remembered as Major Robson, the tall, fine-looking officer with his erect carriage and soldierly bearing, his rare personal magnetism, and, at the end, his military funeral in Berkeley where his brother officers performed their last service for their beloved comrade. To most of his friends he will always be "Ralph Robson", which is as he would have it. To all who knew him, he was an unforgettably gracious friend.

"E'en as he trod that day to God so walked he from his birth,
In simpleness and gentleness and honour and clean mirth."

On June 30th, 1913, Mr. Robson was married in Berkeley, Calif., to Miss Barbara L. Reid, a classmate at the University. He is survived by his wife, his mother, two sisters, and a brother, F. T. Robson, Assoc. M. Am. Soc. C. E. He was a member of the Paso Robles Lodge of Masons.

Mr. Robson was elected a Junior of the American Society of Civil Engineers, on October 3d, 1911, and an Associate Member on June 3d, 1915.

ELIOT NICHOLS SMITH, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1921

Eliot Nichols Smith was born in East Hartford, Conn., on August 6th, 1878, of English ancestry. His father was John B. Smith and his mother, Lucy Nichols Smith. The family was long settled in New England, and Eliot Smith's youth was spent in Connecticut. A great-grandfather named West served in the Revolutionary War; he entered as a private, was afterward made a Sergeant, and, finally, became a surgeon.

Both Eliot Smith's grandfathers were ministers, Grandfather Smith preached only occasionally, being interested in medicine and in farming as well. Grandfather Nichols was a Congregational minister in Connecticut for many years. His salary was only $450 a year, so it was necessary not only that he should practice rigid economy, but that he should raise as much as possible on his few acres. Eliot Smith was thought to resemble this Grandfather Nichols. He had the same instinctive hatred of waste, an equally strong love of hospitality, and a steady, practical intelligence.

His father, John B. Smith, did not have much formal education, but he studied at the Sheffield Scientific School for a few months. He was always prominent in Sunday School and temperance work. He was eager, impulsive, full of new and old ideas, and always bursting with his passionate beliefs. For most of his life, he was a market gardener and had fine orchards where the boys of the family worked during the summer, not always, perhaps, to their complete satisfaction. Indeed, summer was to Eliot Smith, the oldest son, distinctly difficult. He was like his mother in temperament, and the bond between them grew constantly stronger.

In the family group there were six girls and three boys, and many were their good times, with games, inpromptu entertainments, and music. One of the sisters, Laura, went as a missionary to South Africa, and a brother, Harry James, was always interested in writing. He became a well known playwright, and was killed during the World War, while in Red Cross Service.

With a nature strong and fine like that of Eliot Smith, the influence of the different members of the family group with interests so diverse, was not so much specific, as a general broadening of outlook.

His common school education was received at the Model School connected with the State Normal School in New Britain, Conn., and he prepared for college at Williston and afterward at Oberlin Academy, and was graduated from Oberlin College in 1900. At Oberlin, he was able to satisfy, somewhat, his deep love for music. He sang in the choir, and was the leader of the College Mandolin Club. His happiest student days, however, were those at the Lawrence Scientific School, at Harvard, from which he was graduated with honors in 1904.

Mr. Smith's studies at Harvard were interrupted in 1901, when he accepted a railroad position in Pittsburgh, Pa. He contracted typhoid fever while

* Memoir prepared by James F. Sanborn and Robert Ridgway, Members, Am. Soc. C. E.

on this work, from which his recovery was so slow that a trip to Colorado was deemed advisable. His brother Roy, who was a student at that time at Colorado College, writes:

"During my Freshman year I was greatly thrilled at Eliot's visit, you may be sure. We lived together in the same room some of the time, getting our own breakfast, and went off with one or two kindred spirits on long tramps to the various mountains and canyons in the neighborhood. He gained much in health, got a little valuable surveying experience, and returned to Harvard to finish up his course."

His return to Harvard was characteristic. "I do not think I will ever make any more money by going back", he said, "but I do think my life will be better worth living", and he never regretted his decision.

Herbert M. Hale, M. Am. Soc. C. E., writes:

"I first knew Eliot intimately when he and I were Assistant Instructors in surveying at the Engineering Camp at Squam Lake. He had a way with him that made the students under his care really like the work. That was just prior to our Senior year."

The following is quoted from a letter by Dean G. Edwards, M. Am. Soc. C. E.:

"Eliot was graduated with our class in Civil Engineering in 1904 and very pleasant relations were established among us in Cambridge. Eliot took the camp courses at Squam Lake in the summer of 1903, and I remember particularly, as a great event, the long hike and climb of Mt. Whiteface over the 4th of July of that year. Eliot was one of our party of twelve bunking with us in a barn one night before the Fourth, and carrying his share of the baggage and enduring the blistered feet with the good nature that made the party a success and the occasion memorable."

After his graduation from Harvard in 1904, Mr. Smith went to New York City and began work as an Assistant Engineer on the new subways, remaining in the service of the Rapid Transit Railroad Commission until the spring of 1906. Mr. Edwards writes:

"A group of us met again in New York after graduation in August 1904, when we entered the employment of the Rapid Transit Commission. Seven of us, Eliot Smith, Sprague, Hogan, Hale, Hanavan, Gilman, and myself leased an apartment on 118th Street, opposite Columbia and lived there together for one year." * * * "He was a lover of music—introduced me to the opera—and played the violin in our 'apartment orchestra', with Hale and Gilman as assisting artists, back in 1904 and 1905."

In 1906, Mr. Smith entered the employ of the New York Board of Water Supply and continued on the Catskill Aqueduct work until 1914. The following is quoted from a letter by John P. Hogan, M. Am. Soc. C. E.:

"Eliot was one of the few who passed all three examinations for the Board of Water Supply with flying colors; he was, therefore, one of the first to go up on the Board of Water Supply work, and if my memory is not at fault, was with Sanborn both at Hurley and at High Falls. He was then in New Paltz for a while, and was graduated from there to the Poughkeepsie office, coming back to High Falls as my assistant after White left. He was a very hard worker and had an exceptionally keen and analytical mind. He was very actively concerned in the location of the Esopus cut-and-cover on the estimating side. We all know that he was personally of a very fine character,

absolutely honest, both morally and mentally, with a very high ethical standard. He could be counted on for not only official loyalty, but also for very strong personal loyalty. He had pleasant manners, a ready wit, and keen sense of humor. My associations with him were always most pleasant, and I learned to admire and respect his high gifts of character and intellect."

On this work, Mr. Smith was at first engaged as Assistant Engineer on location studies for the Esopus and Wallkill Divisions and was stationed at New Paltz, N. Y., for about a year. He was then transferred to the Department Office, at Poughkeepsie, N. Y., and was employed on a great variety of problems concerned with the location studies and estimates and the preparation of information for the construction contracts for the sixty miles of aqueduct of the Northern Aqueduct Department.

In 1911, Mr. Smith was assigned to the Esopus Division as Assistant to the Division Engineer, and remained in this position about a year. During several months of this period, he was in charge of the engineering work of the Division, which included the first twelve miles of the Aqueduct below the Ashokan Reservoir and which cost about $9 000 000 to construct. On the practical completion of the work on this Division in 1912, Mr. Smith was transferred to the office of the Northern Aqueduct Department, which had been moved from Poughkeepsie to Cornwall-on-Hudson, acting as Assistant to the Department Engineer. During the time he was in Cornwall, practically all the work in the Northern Aqueduct Department was brought to a successful completion. The writers are familiar with Mr. Smith's ten years on the Catskill Aqueduct and know how much of himself he put into this work.

In 1914, feeling that the work of the Board of Water Supply was nearing completion, and that his best efforts would be called forth by some new piece of work in engineering, Mr. Smith resigned from the Board, and took up his residence in Nutley, N. J. For several months, he was engaged on the preparation of contracts and designs for a number of projects in the vicinity of New York City, notably a new pipe line for Jersey City, N. J.

During these years, he began more and more to identify himself with the first interests of the community in which he lived, a longing which his constant change of residence heretofore, had given little opportunity to satisfy. He became a member of the Board of Trade of Nutley, was for four years leader of a troop of Boy Scouts connected with Grace Protestant Episcopal Church, building it up from a sparse, uninterested membership to a flourishing organization for town service, and served as Treasurer on the Campaign Committee which helped to elect Emil Diebitsch, M. Am. Soc. C. E., as Mayor of the town.

Mr. Smith was appointed to be the first Town Engineer of Nutley, on July 10th, 1916, and served the community in that capacity for more than three years. He was placed at the head of the newly created Bureau of Engineering and had responsible charge of the inspection of buildings under construction and the issuance of building permits, the maintenance of roads and sidewalks and the laying of all new pavements, the maintenance of the water system, the extension of mains, the making of house connections and the reading of meters, and the construction of a sewer system covering the entire town and estimated at that time to cost $350 000. Starting with less than

nothing, with patience, perseverance, and industry, Mr. Smith soon systematized the activities placed in his charge and developed an effective organization which functioned intelligently and economically. By introducing the latest methods and processes in the repair and resurfacing of the macadam roads, he greatly improved the highways without increasing the cost of maintenance.

The work of municipal engineering, with all its complex phases, was particularly appealing to Mr. Smith. He liked the personal contact with fellow townsmen, the frequent necessity of delving into legal matters, and the diverse municipal questions connected with economics, a study in which he was particularly sound. A great aspiration of his was to serve some community in the capacity of City Manager.

In 1919, to the regret of the Mayor and all others acquainted with his work, Mr. Smith resigned as Town Engineer and moved to Lisbon, N. H., where he bought a home and engaged in the wood-turning business.

He was married in 1909 at Keene, N. H., to Miss Margaret Pollard who was an Instructor of English at Vassar College, and a woman of unusual quality. They had four children, two girls and two boys.

Mr. Smith's sister Edith writes:

"There was a singular unity about him, an utter freedom from self-consciousness in all he did. If it was his job, he did it with a uniform standard of excellence and thoroughness."

> "He scarce had need to doff his pride or slough the dross of earth,
> E'en as he trod that day to God so walked he from his birth,
> In simpleness and gentleness and honour and clean mirth."

Mr. Smith was elected a Junior of the American Society of Civil Engineers on April 2d, 1907, and an Associate Member on June 30th, 1910.

CHARLES WHITING BRADLEY, Assoc. Am. Soc. C. E.*

DIED JANUARY 14TH, 1921.

Charles Whiting Bradley was born at Newtown, Conn., on September 13th, 1836.

He entered the service of the Erie Railroad Company at Nanuet, N. Y., in November, 1852, as Telegraph Operator, and remained with that Company until 1862. From 1862 to 1868, Mr. Bradley held several positions with the Atlantic and Great Western Railroad Company, rising to that of Division Superintendent.

From 1868 to 1870, he served as Superintendent of the Cincinnati, Chicago, and Louisville Railroad, and from 1870 to 1872, he was engaged as Superintendent of the Niles and New Lisbon Railroad. In 1872, he returned to the Atlantic and Great Western Railroad as Division Superintendent, which position he held until 1873. From 1873 to 1878, Mr. Bradley was employed as Division Superintendent of the Wabash, St. Louis, and Pacific Railway, and, from 1878 to 1880, he served as General Western Traffic Manager of the same road.

In 1880 and 1881, he was Division Superintendent of the Denver and Rio Grande Railroad, and from 1881 to 1882 he was Superintendent of the Cincinnati and Northern Railroad.

During 1882 and 1883, he held the position of Assistant Superintendent of Construction with the New York, West Shore, and Buffalo Railway Company, and from 1883 to 1885, he was Division Superintendent of that road. From 1885 to June 1st, 1898, Mr. Bradley was General Superintendent of the West Shore Railroad, and, afterward, was connected with the Walkill Valley Railroad, and also served as Manager of the Western New York Car Service Association, at Buffalo, N. Y.

Mr. Bradley then entered the service of the Chesapeake and Ohio System as Superintendent of the Telegraph Department, which position he held until his death on January 14th, 1921, at Nyack, N. Y. For ten years before his death he had made his home in Richmond, Va., and was on a visit to his nephew at Nyack where he was taken ill and died.

Mr. Bradley was of modest disposition, kind and gentle, and had many friends. He was married in 1857 to Miss Annie E. Tallman, of Nanuet, N. Y., who survives him.

He was widely known among railroad men and had been for many years a prominent member of the American Railway Engineering Association, having served as a delegate to the International Railway Conference in London, England, in 1895. He also attended a similar convention in Berne, Switzerland, in 1910. At the time of his death, Mr. Bradley was a member and President of the Old-Time Telegraphers' Association of the United States. He was also a member of the Chesapeake and Ohio Veteran Employees' Association and the Westmoreland Club of Richmond, Va.

Mr. Bradley was elected an Associate of the American Society of Civil Engineers on June 19th, 1891.

* Memoir compiled by the Secretary from information on file at the Headquarters of the Society.

GEORGE LORD BURROWS, Affiliate, Am. Soc. C. E.*

DIED NOVEMBER 9TH, 1921.

George Lord Burrows was born in Albion, N. Y., on August 30th, 1836. He received his education at the Albion Academy and Fort Edward Collegiate Institute, where he studied engineering.

After leaving college, he worked with the Engineering Corps on the enlargement of the Erie Canal and afterward with the Bank of Albion, where he gained the knowledge of banking which he used to such good advantage later.

In 1862, Mr. Burrows went to Saginaw, Mich., where his successful career was closely connected with the history of the Saginaw Valley.

In May, 1862, he established the private bank of George L. Burrows and Company in Saginaw. This bank was his first business enterprise in that city, and was conducted by him and his associates until 1915, when it was sold to the Bank of Saginaw.

Mr. Burrows saw the possibilities of the pine lands in Michigan, and early became prominently identified with the lumber interests of Central Michigan, in the development of which he became a powerful influence.

About the same time, he became interested in the construction of the first street car line in Saginaw. He served as Treasurer of the Company which built the line, and also of the Company which built the Mackinaw Street Bridge.

In 1885, Mr. Burrows was elected President of the Niagara Falls International Bridge Company. His father, Lorenzo Burrows, had been one of the builders of the Suspension Bridge across the Niagara River near the Falls, and Mr. Burrows succeeded his father as head of the organization controlling the bridge. He held this position at the time of his death. In 1897, during his incumbency of this office, the Suspension Bridge was changed to a steel arch bridge.

As a lumberman, banker, and engineer, Mr. Burrows was closely identified with many of the early enterprises of Saginaw, and did much toward the development of the city, to which he gave freely of his services in its public works. He served as Chief of the Volunteer Fire Department and in this capacity superintended the construction of the West Side Water-Works. He also acted as Supervising Engineer on the construction of the Saginaw County Court House, and was for many years a member of the School Board of the West Side Union School District.

He was regarded by his friends and acquaintances as a man of unusual intelligence and integrity, well informed, and an excellent financier. He gave generously of his wealth, both to individuals and organizations, and always insisted that nothing be said concerning his charities.

* Memoir compiled by the Secretary from information on file at the Headquarters of the Society.

The following high tribute has been paid to Mr. Burrows:

"It may be said of him without reserve what cannot be said of every man who has acquired large means, his best friends are those who know him best and his personal integrity is without stain."

Mr. Burrows was married on June 25th, 1863, in Buffalo, N. Y., to Miss Julia S. Hotchkiss, who died in Saginaw on October 14th, 1883. He is survived by one son and five daughters.

He was a member of the Engineers' Club, the Union League Club, and the Bankers' Club, all of New York City, and of the Biscayne Bay Yacht Club, of Miami, Fla. He also belonged to the Masonic Order.

Mr. Burrows was elected an Affiliate of the American Society of Civil Engineers on February 3d, 1886.

JAMES RICHARD DONALD MACKENZIE, Assoc. Am. Soc. C. E.*

DIED JANUARY 25TH, 1921.

James Richard Donald Mackenzie, was born in Blackburn, Lancashire, England, on December 25th, 1880. When he was a small child, his parents took up their residence in Canada, where he received his early education, and, later, moved to Massachusetts where his education was completed.

Mr. Mackenzie's first experience in the business world was in May, 1906, when he went to Oakland, Calif., and became connected with the Watson Roofing Company. He soon became Manager of that Company, and, later, was made a partner, in which capacity he ws in direct charge of the design and construction of numerous paving, roofing, and water-proofing jobs, involving the use of various grades of bitumens. Mr. Mackenzie severed his connection with the Watson Roofing Company in November, 1907, and established a business of his own in the same line of work.

In 1916, due to a combination of economic conditions and personal affairs, Mr. Mackenzie closed his business in California and moved to Kansas City, Mo., where he became connected with the Kansas City Branch of the Barrett Company. His knowledge of bituminous materials and their derivatives made him a valuable asset to the Sales Organization of this Company, and the major portion of his time was spent in the promotion of paving materials in the State of Kansas.

Mr. Mackenzie had a wide acquaintance among paving contractors and engineers in Kansas, and his opinions and advice were always reliable. In the years that he had followed water-proofing work, he became an authority on the subject. Having always been a thorough student and an intelligent thinker, his studies had led him deeply into the consideration of the whole subject of water-proofing.

Mr. Mackenzie died on January 25th, 1921, in Tucson, Ariz., from an acute attack of pneumonia. He is survived by his widow and a daughter, Margaret, who reside in Kansas City, Mo.

Mr. Mackenzie was elected an Associate of the American Society of Civil Engineers on June 1st, 1920.

* Memoir prepared by Arthur B. Hitchcock, Assoc. M. Am. Soc. C. E.

CALVIN TOMKINS, Assoc. Am. Soc. C. E.*

DIED MARCH 13TH, 1921.

Calvin Tomkins, the son of Walter and Emma Augusta (Baldwin) Tomkins, was born in East Orange, N. J., on January 26th, 1858.

He was graduated with honor from Cornell University, in the Class of 1879, with the degree of A. B. Within a few years thereafter he was elected President of the New York Association of Cornell University, and throughout his life was a leader among Cornell Alumni and active in support of the University.

Early becoming one of a group of art workers and amateurs, Mr. Tomkins was associated with leading artists of New York City in the movement from which later grew the Fine Arts Federation, with its associated societies. He became President of one of the more prominent of these societies, the Municipal Art Society, and was also one of the founders of the National Arts Club.

In 1887, he assisted in the organization of the Reform Club, and was successively prominent in its Tariff Reform, Sound Currency, and City Development Committees. He also served for years as President of the Club.

Having been active in the re-election of President Cleveland, whose confidence he enjoyed, he was a leader in the Gold Democratic Movement in opposition to Free Silver, and a candidate for Congress in the Palmer and Buckner Campaign.

In 1884, he succeeded his grandfather, Calvin Tomkins, as a Director in one of a group of business concerns which since early in the last century had been developed by members of his family, and the death of his father in 1896 left him mainly responsible for the conduct of manufacturing and mining plants at Tomkins Cove and Rondout, N. Y., Newark, N. J., and in Nova Scotia and New Brunswick, Canada.

Always a student of the problems growing therefrom, Mr. Tomkins was early recognized as an expert by the New York Chamber of Commerce, Board of Trade and Transportation, Maritime Association, and other commercial bodies of New York City, in which, before 1910, he had become prominent in helpful criticism of subway development, and as an adviser of Governor Hughes in the reform by which the old commission was displaced and plans progressed for city ownership and control.

Having been active and influential in securing the passage of the Constitutional Amendment exempting self-sustaining dock and subway bonds from the debt limit of New York City, and a leading advocate of the election of Mayor Gaynor, he was urged by the Mayor, on his election, to share the responsibility of the Administration, and notwithstanding business obligations which he could not transfer, he accepted the office of Commissioner of Docks on January 1st, 1910, an appointment which gave great satisfaction to the commercial interests of the city.

* Memoir prepared by B. F. Cresson, Jr., M. Am. Soc. C. E.

Mr. Tomkins commenced and developed vigorous and tactful propaganda for adequate development of the Port of New York as a unit, irrespective of city and State boundaries. This propaganda has gradually enlisted public support, until the essential of the plan urged by Mr. Tomkins is the groundwork of the world port to which the States of New York and New Jersey are practically committed.

Mr. Tomkins was not in sympathy with the policy of executing long-term leases of docks and piers, and most of the leases which fell due during his administration were held over on permits for one or two years, in an effort to secure control of considerable stretches of water-front and institute a comprehensive physical plan.

He sought information as to port administration, organization, and equipment, not only at American, but also at European, ports, and was a strong advocate of public control of harbor facilities, such control to extend at least as far as public ownership.

He was insistent in his demand that there must be a railroad connection between the west side and the east side of the port, for the movement of railroad freight, by tunnel or bridge. Plans were developed by him for bringing all the railroads into Manhattan to joint railroad terminals; and legislation secured to make possible the construction of a marginal railroad in Brooklyn, and in developing plans for many sections of the port, took into consideration the needs and possibilities of New Jersey as well as New York.

In 1911, Mr. Tomkins called together representatives of the Port Authorities in the United States, and at a meeting held in the Chamber of Commerce of the State of New York, he organized the National Association of Port Authorities, of which he was elected President, the Association later changing its name to the American Association of Port Authorities, in order to include the port authorities of Canada.

The Material Handling Machinery Manufacturers Association, an organization made up of the manufacturers of freight-handling machinery, organized primarily at the request of the United States Shipping Board early in the World War, chose Mr. Tomkins as its President.

He was called on by the United States Government during the World War and served as a member of the Board devoted to the operation of inland waterways of the country, among the activities of which was included the operation of the New York State Barge Canal.

His co-operation and advice were sought by the authorities of many of the ports of the country, and his judgment was held in high esteem in harbor, terminal, and transportation matters. Acting on his advice, the City of Wilmington, Del., acquired a large tract of property and has commenced the construction of a municipal terminal.

Mr. Tomkins was a member of the Commission appointed in 1911 by the Governor of New York State to co-operate with a similar Commission appointed by the Governor of New Jersey to investigate conditions at the Port of New York and to make recommendations.

He was a Delegate to the National Democratic Convention in San Francisco, Cal., in 1920. He was taken seriously ill on the trip, and although he

partly recovered, he never quite regained his full health, and died on March 13th, 1921.

He was married on December 4th, 1889, to Kitty Neilly, of Stony Point, N. Y., who survives him.

His death is a severe loss to the city in which he lived, to the transportation interests, and to those who have the responsibility of creating and improving the port terminals of the United States.

Mr. Tomkins was elected an Associate of the American Society of Civil Engineers on January 6th, 1886.

JAMES FRANCIS WRENN, Affiliate, Am. Soc. C. E.*

DIED NOVEMBER 2D, 1921.

James Francis Wrenn, the son of James and Sarah Wrenn, was born at Lynchburg, Va., on May 30th, 1867. He received his education at the public schools of his native town, and was graduated from the Virginia Military Institute.

Mr. Wrenn's first experience in engineering work began at the early age of fifteen. Owing to ill health, he was compelled to discontinue his studies temporarily, and on February 2d, 1882, went to Alabama with an uncle who was a prominent railroad contractor. At that time his uncle had a contract for the construction of 20 miles of the Georgia Pacific Railroad, between Atlanta, Ga., and Birmingham, Ala. The boy was employed on this work until September, 1882, when he returned to Lynchburg to resume his studies. In September, 1887, he entered the Virginia Military Academy. During his summer vacations, Mr. Wrenn worked for his uncle and for his grandfather, who was also a contractor, on their different construction contracts.

In 1890, immediately after his graduation from the Institute, he was appointed Assistant Superintendent and Engineer of Construction on a contract for double-tracking the Norfolk and Western Railroad, west of Salem, Va.

From 1892 to August, 1893, Mr. Wrenn was engaged as a Contractor, by the Roanoke Development Company, to build a cantilever bridge across the Roanoke River, at Roanoke, Va., and to construct the first improved street for the City of Roanoke.

From August, 1893, to September, 1894, he constructed five miles of new grade for the Atlantic Coast Line Railroad Company, on the Augusta Cut-off around Petersburg, Va., from Dunlap to Rheme Stations.

From March, 1895, to April, 1897, Mr. Wrenn was engaged in the construction of a canal and dam for developing a new water power for the Roanoke Rapids Power Company, at Roanoke Rapids, N. C., and from April, 1897, to March, 1898, he constructed ten miles of the Mobile, Jackson, and Kansas City Railroad, between Mobile, Ala., and Hattiesburg, Miss.

From March to July, 1898, he was employed in lengthening the sidetracks on the Mobile Division, at Flemington, Ala., for the Louisville and Nashville Railroad Company, and from July to December, 1898, in constructing two miles of new grade for the Seaboard Air Line, between Lawrenceville and Logansville in Georgia, thirty miles east of Atlanta.

In 1899, Mr. Wrenn was appointed General Superintendent of Construction for Samuel Walton, Contractor, and was engaged in constructing for the Norfolk and Western Railroad a low-grade line cut-off, near Radford, Va., which included a tunnel through a spur of the Blue Ridge Mountains. He was also engaged on the construction of the last ten miles for the Virginian and Southwestern Railroad, from Elizabethton to Mountain City, in Tennessee.

In 1900, he was employed on the subway construction in New York City, and, in 1901, as a Contractor on the construction of a soft tunnel for the

* Memoir prepared by S. A. Lovette, Esq., Roanoke, Va.

Government, in Washington, D. C., during which a large intercepting sewer was laid 60 ft. under the surface of M Street, S. E. In 1902, he constructed an 18-mile electric line from Danbury, Conn., to Golden Bridge, on the Harlem Division of the New York Central Railroad, in New York State, and, in 1903 he built 20 miles of electric line from Rutland to Fair Haven, in Vermont.

In 1904, Mr. Wrenn was made General Superintendent for a contracting company, in which capacity he constructed a section of the Wabash Railroad into Pittsburgh, Pa. In 1905, he became General Superintendent for the William J. Oliver Company, of Knoxville, Tenn., and was engaged on the work of double-tracking the "Big Four" Railroad between Cincinnati, Ohio, and Indianapolis, Ind. In July of that year, he was transferred to Norfolk, Va., to take charge of the construction of the first 100 miles of the Tide Water Railroad (now the Virginian Railway), from Norfolk, Va., west.

In July, 1906, he was sent to New Orleans, La., in charge of the construction of the first 100 miles of the New Orleans and Great Northern Railroad, from Slidel, La., toward Jackson, Miss. In 1907, Mr. Wrenn was appointed General Superintendent for a Philadelphia company, and had charge of the construction of a low-grade line of 25 miles for the Erie Railroad, in New York State, from Hornellsville on the Buffalo Division, to Guba, on the Chicago Division. This cut-off was constructed for the purpose of eliminating heavy grade on the Chicago Division, and was an extremely difficult piece of work.

During 1908 and 1909, he held the position of General Superintendent for a New York contracting company and was engaged in building improved highways in New York State, west of Albany, at Canajoharie, Fonda, Gloversville, and Northville. This work was done under the first $50 000 000 appropriation made by the Public Works Department of New York State. In 1910, he was transferred and made General Manager of two sections of the New York State Barge Canal on the Champlain Division, between Waterford and Stillwater, N. Y.

In 1911, Mr. Wrenn formed a co-partnership with Mr. F. G. McGuire of Norfolk, Va., which was incorporated as the McGuire Construction Company, and served as its Vice-President and General Manager until March 31st, 1916. During this period this Company constructed improved streets for the Cities of Fayetteville, Newbern, Wilmington, and Greensboro, N. C., nine miles of concrete highway in Prince George County, for the State of Maryland; laid asphalt paving for the Towns of Easton and Salisbury, Md.; and constructed asphalt streets for the City of Danville, Va.

In 1916, Mr. Wrenn bought out Mr. McGuire's interest, and continued to operate the Company as President and Manager. During the same year, he constructed asphalt streets for the City of Suffolk, Va.

During the World War, he gave his services to the Government, and afterward was engaged as Constructing Engineer for the R. D. Lassiter Company, of Norfolk, Va.

Mr. Wrenn was very popular in the business as well as the social circles of Norfolk, Va. He is survived by one daughter, Virginia, ten years of age.

Mr. Wrenn was elected an Affiliate of the American Society of Civil Engineers on September 6th, 1905.

FREDERIC BORRADAILE PRICHETT, Jun. Am. Soc. C. E.*

DIED SEPTEMBER 6TH, 1918.

Frederic Borradaile Prichett was born at Philadelphia, Pa., on November 23d, 1890, the son of William Borradaile and Fannie W. Prichett.

After completing his elementary grammar school work, he entered the William Penn Charter School in Philadelphia, on September 16th, 1904. From the time of his entrance until his graduation in 1909, he was identified at all times with the leading activities of the school, taking an active interest in athletics, and in debating and literary activities. He was Editor in Chief of the *Penn Charter Magazine,* the school publication, President of the A. D. Gray Science Club, Manager of the Cricket Team, a member of the School Debating Team, Vice-President of the Literary Society, and a member of the Mandolin and Glee Clubs, and also of the School Orchestra. He was at all times an able and persuasive speaker and was one of three chosen at large from the school to compete in the prize declamation contest at the annual entertainment in the spring of 1909. He was one of the Commencement speakers and also Historian of his class.

He was a member of Tau Theta Sigma Fraternity of the Penn Charter School, and, in his senior year, was elected a member of Alpha Delta Tau, an honorary society among preparatory schools, corresponding to Phi Beta Kappa.

In the fall of 1909, he entered the Engineering School of the University of Pennsylvania, and in October of the same year became a member of Phi Delta Theta Fraternity. During his career at the University, "Ted" Prichett, as he was known among his friends, was intimately associated with many activities. He took an active interest in social welfare work from the time of his entrance and, in his Senior year, was elected President of the Young Men's Christian Association of the University. In his Senior year, he was appointed Manager of the University basket-ball team, was Valedictorian of his Class, and served on numerous Class Committees. He was a member of the Friars Senior Society, the Plumb Bob Society, and the Civil Engineering Society of the University, and in June, 1913, was graduated with the degree of B. S. in C. E.

During the summer, while attending the University, Mr. Prichett was interested in the practical phases of his profession, working with the Pennsylvania Railroad Company as Rodman and Surveyor and with Hale and Kilburn as Draftsman.

On his graduation from the University of Pennsylvania, he became associated with the Pennsylvania Railroad Company, in the Maintenance of Way Department, and was employed by that Company until the spring of 1915, when he entered the employ of Gibbs and Hill, Engineers, then engaged in the electrification of the Chestnut Hill Branch of the Pennsylvania Railroad. On May 18th, 1916, he was married to Miss Gertrude Bailey Rhoads, of Philadelphia, Pa. In the fall of 1916, he went to Cincinnati, Ohio, in the employ of

* Memoir prepared by W. B. Prichett, Esq., North Wynnefield, Philadelphia, Pa.

the American Bridge Company, remaining there until the outbreak of the World War.

In June, 1917, Mr. Prichett returned from Cincinnati and enlisted as a Private in the First City Troop of Philadelphia, and in August went South with that organization to Camp Hancock, Augusta, Ga. On January 5th, 1918, he was assigned to the Third Officers' Training Camp, 28th Division, Camp Hancock, Georgia, and on April 19th, was graduated as one of twenty-six officers from the Division commissioned in the Field Artillery. He was immediately assigned to Headquarters, 109th Field Artillery Regiment. On May 18th, Lieut. Prichett sailed overseas with that regiment arriving at Liverpool, England, on May 31st, 1918. On June 10th he entered on his final course of artillery training at Camp Meucon, France, and was assigned to Battery A, 109th Field Artillery. On August 6th his regiment entrained for the front and from August 11th until his death on September 6th, 1918, he served with distinction as Junior Officer in his battery, on many occasions receiving unstinted commendation from his superior officers for technique in firing his battery and his coolness under enemy fire.

On September 4th, 1918, the 28th Division, as a unit of which his battery was serving, was ordered to cross the Vesle River in the vicinity of Fismes. His battery was ordered to advance in the face of direct enemy observation. On the morning of September 5th, his battery was subjected to a terrific barrage. Refusing to seek cover until all his men had first received protection, Lieut. Prichett was wounded by shell fire, from the effects of which he died the following day, September 6th, 1918. He was buried at Coincy, France, but his body subsequently was removed to the now famous National Cemetery at Belleau Woods, near Chateau Thierry.

Lieut. Prichett was elected a Junior of the American Society of Civil Engineers on May 15th, 1917.

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